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ERRATA.

Page 57, sixth line. For "firmness" read "fineness."

Page 91, thirty-fifth line. After "surface," add "a layer of brick laid flat and dipped in asphalt, with proper precautions, or"

Page 94, thirty-seventh line. "Cold liquid asphalt paint" is a relative term used to distinguish paint heated only enough to make it liquid from the rock asphalt raised to a temperature of 300° and used as the outside coat.

Page 128, sixteenth line. For "paris green" read "plaster of paris."

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No. 770.

EFFECT OF DEPTH UPON ARTIFICIAL WATERWAYS.

By THOMAS C. CLARKE, M. Am. Soc. C. E.

PRESENTED OCTOBER 16TH, 1895.

WITH DISCUSSION.

In examining the question of deep waterways between the lakes of this country and the ocean, the author has collected some facts of a general engineering interest, which he now presents to the Society. The question of first cost, the interest on which is represented by tolls, will be disregarded, and the inquiries will be confined to ascertaining the cost of transportation alone, taking the case of the Upper Lakes and Erie Canal between Chicago and New York as an example.

To ascertain the cost of transportation on this system these data are necessary :

First.—The number of trips which steamers and barges can make during the season of navigation. This depends upon the possible speed in artificial waterways. The number of trips per season also depends upon some other matters which do not affect this discussion; such as the number and character of the locks, the length of detention in port, etc.

Second.—The yearly cost of operating and maintaining large steamers, also of steam-towed barges.

Third.—The receipts per trip must be equal to the amount of annual expenses divided by the number of trips, to prevent loss. For the sake of comparison the amount of receipts per trip will be equally divided between the receipts from grain going east, and merchandise going west. The cost of carrying grain will be ascertained by multiplying the number of bushels carried per trip by a rate which will make up a half of the expenses per trip. Such rates will always be a little less than the actual rates, as shippers must provide for the contingencies of light cargoes. As the rule is equally applied to all the conditions of depth, the comparison will be a fair one.

Transportation between Lakes Superior, Michigan, and Erie costs less now than transportation by long ocean voyages, as the vessels have full cargoes both ways of grain, coal, ore, lumber and merchandise. They do a local business in addition to the through business, which is not possible on the ocean. Hence, people say, "If we could only get a ship canal so that lake vessels could go to New York without breaking bulk, the economy of transportation would be extended nearly 500 miles farther." It really means not only a ship canal, but a ship canal of such dimensions that the expensive ship shall not be detained too long while passing through it.

The conditions will now be examined. All of these are known except that of the depth of water under the keel of the ship, necessary to enable her to move at the required speed. If the question were simply one of resistance it could be solved mathematically, but there are other things to take into account. The Suez Canal when opened was 27 ft. deep, and vessels loaded to 25 ft. draft are able to pass through at the rate of 4½ miles an hour. If they try to move faster, the currents of water passing under their bottoms to replace that driven away by the screw, cause them to lose steerage way, and the ship takes a shear, runs her nose into the bank and stops not only herself, but other vessels. Greater width would help. The Suez Canal is now being dredged 5 ft. deeper and widened. It is hoped that speed can be increased to 6 or 7 miles an hour when the canal is deepened to 32.5 ft.

As a type of a lake freighter, the new steamer *Victory* will be taken. She is 380 ft. long by 48 ft. beam, and carries a cargo of 4 000 net tons

at 14 ft. draft; 5 000 tons at 16 ft., 6 000 tons at 18 ft., and 7 000 tons at 20 ft. Her economical rate of speed on the Lakes in deep water is 13 miles an hour.

The depth of lake harbors now contemplated by the United States Government is 20 ft. No port but South Chicago has 18 ft., the rest having not much over 16 ft.

Half an hour will be allowed for lockages in a ship canal, and the same for a fleet of barges. Thirteen days will be assumed as the time of detention in port in all cases. This time seems long, but is necessary in order to do a local business and get full cargoes. Better organization and machinery for handling freight would shorten this time.

Ship Canals.

I.—Erie Canal deepened to 18 ft. to admit steamers carrying 5 000 tons on 16 ft. draft. Distance: Chicago to Buffalo, by Lakes, 870 miles; Hudson River, 143 miles; total, 1 013 miles. At 13 miles an hour; time, 78 hours. Canal, 350 miles; speed per hour, 4 miles; time, 87.5 hours. Lockages, 72 reduced to 50, at one-half hour each, 25 hours. Total time in motion, 190½ hours. Round trip, 381 hours, or 16 days. Time in port, 13 days. Total round trip, 29 days, or seven trips per season of seven months.

Estimated Expenses.—210 days at $250	$52 500
Insurance and other expenses	43 950
Interest, 10% on cost, $200 000	20 000
	$116 450

Estimated Receipts per Trip.—One-seventh of $116 450 is $16 635, half of which is $8 318. The ship carries 5 000 tons on 16-ft. draft, equal to 166 500 bushels of wheat, the rate on which would be 5 cents; this would be $8 325 on 166 500 bushels.

II.—Erie Canal deepened to 25 ft., to admit steamers carrying 7 000 tons burden. Speed increased on canal possibly to 7 miles an hour. Distances: Lakes and rivers, 1 013 miles, at 13 miles an hour, 78 hours. Canal, 350 miles; at 7 miles an hour, 50 hours. Lockages, 25 hours. Total, 153 hours; round trip, 13 days. Time in port, 13 days, or 26 days, eight trips per season.

Estimated Expenses.—Allowing for extra coal, $120 000.

Estimated Receipts per Trip Going East.—One-eighth of $120 000 is $15 000, and one-half of that is $7 500. Ship carries 7 000 tons or

233 000 bushels, which, at 3.22 cents per bushel, makes $7 500, Chicago to New York.

BARGE CANALS.

Cost by Lakes to Buffalo and by present Erie Canal, 7 ft. deep.

III.—The present cost of conveying grain from Chicago to Buffalo is ..... 1.25 cents.

Elevating and trimming cargo at Buffalo	1.15	"
Canal to New York	2.33	"
Chicago to New York	4.73	"

Actual rates from 4¾ to 7 cents.

IV.—Cost by Lakes to Buffalo and by Erie Canal deepened to 9 ft. Fleets of one tug and five steel barges loaded to 7½ ft. draft. Cargo, 1 500 tons.

Erie Canal, 350 miles. Speed, in motion, 4 miles an hour. Time, 87½ hours. Lockages, all locks lengthened for two boats, 15½ hours. Hudson River, 143 miles; speed, 6 miles an hour; time, 24 hours. Time in motion, 127 hours. Round trip, 254 hours, or 10.6 days; time in port, 12 days; total, 22.6 days round trip. Number of trips in 210 days, nine.

Estimated Expenses.—210 days, at $30	$6 300
Insurance and other expenses	7 700
Interest on cost, 10% on $40 000	4 000
Total expenses	$18 000

Estimated Receipts.—Full cargoes grain going east, 50 000 bushels at 2 cents is $1 000 ; going west, 1 000 tons at $1, $1 000. Total, $2 000 by nine trips, is $18 000. Total cost of conveying wheat from Chicago to New York *via* Buffalo and Erie Canal, when deepened to 9 ft., is:

Chicago to Buffalo		1.25 cents.
Elevating	.875 cent.	
Trimming	.275 "	1.15 "
Transportation (canal)		2.00 "
		4.40 cents.

An examination of the cost in the third and fourth cases shows that one of the heaviest charges is the 1.15 cents per bushel due to transshipment. Hence the loud cry for ship canals.

V.—Is it not possible that less expensive machinery can do the business on lake and canal? If the insurance companies will insure

a fleet of five steel barges 90 x 17½ ft. and loaded to 6 ft., from Buffalo to Cleveland, 174 miles, would they not insure a fleet of four steel barges 180 x 17⅕ ft., loaded to 7½ ft. draft, from Buffalo to Chicago? Such a fleet could pass the Erie Canal deepened to 9 ft. and with its few short locks lengthened.

The time would be as follows: Lakes and river, 1 013 miles at 6 miles an hour, 169 hours; canal, 350 miles at 4 miles an hour, 87.5 hours; lockages, 15.5 hours; total in motion, 272 hours × 2 = 544 hours or 23 days. Time in port, 13 days, or 36 days per round trip, allowing six round trips per season.

Expenses calculated on same basis as before, estimated at $24 000, to cover larger mileage and more coal used.

Estimated Receipts.—One-sixth of $24 000 is $4 000, and one-half is $2 000. They carry 2 400 tons, or 80 000 bushels, and the rate should be 2½ cents, to make $2 000.

It seems to the author that the proper thing is for the people of New York to vote for the 9-ft. enlargement; and the fleets of barges running on both Lakes and canal must follow. The profit would be so great that a very high rate of insurance could be paid.

SUMMARY OF COST OF TRANSPORTATION, CHICAGO TO NEW YORK.

I.	Ship canal 18 ft. deep, no transshipment.....	5 cents.
II.	" 25 " " "	3.22 "
III.	Barge canal and Lakes, 7 ft. deep, including transshipment.....	4.73 "
IV.	Barge canal and Lakes 9 ft. deep, including transshipment.....	4.40 "
V.	Barge canal and Lakes, no transshipment.....	2.5 "

The object of this paper, besides the usual one of exciting friendly discussion, is to call the attention of the profession to what seems to be proved in the foregoing, that a ship canal ought to be not less than 25 ft. deep, if it be intended to pass large lake vessels; and that a slight enlargement of the Erie Canal may allow fleets of steel barges to run from New York to Chicago, and transport freight at less rates than large lake vessels could do in a 25-ft. ship canal, even if it were used free of tolls. If this be true, why are ship canals wanted? In the judgment of the author, there is but one such canal needed; a ship canal around Niagara Falls on our own territory, allowing upper lake vessels to reach Lake Ontario ports, and there transship by rail or canal to New York.

DISCUSSION.

Mr. North. EDWARD P. NORTH, M. Am. Soc. C. E.—While there probably will be substantial unanimity in the indorsement of that part of the paper which declares for a canal of not less than 25 ft. depth, if intended to pass lake vessels, it may be worth while to examine the strongly urged alternative—a canal of no greater width of locks than at present, with only 8 ft. on its mitre-sills and 9 ft. in its fairway. This canal, according to the present State Engineer, Mr. C. W. Adams, will allow an increase of tonnage from 250 to 300 tons per boat, a matter of one-fifth; but as all of the locks are to be so lengthened that they will chamber two boats, the theoretical increase of business to be done on the canal will not be from 3 144 144 to 3 772 973 tons, but to about 7 500 000 tons. This would be an increase of some 60% over the tonnage of 1880, the year of greatest traffic. But the same forces which have reduced the tonnage on the Erie Canal from 4 608 651 tons in 1880 to 3 144 144 in 1894 will doubtless continue in operation, and it is probably safe to predict that the canal enlarged as urged will not deliver 5 000 000 tons of western and local freight to the city of New York any two consecutive years.

The proposition is in effect to maintain a stricture 350 miles long between the 40 000 000 or 50 000 000 tons of lake transportation and the 60 000 000 or 70 000 000 tons of shipping that crosses the harbor and bay of New York—a narrow and inadequate channel, through which the 5 000-ton cargoes of lake vessels are to be dribbled in 300-ton lots to meet 10 000-ton sea carriers. Shippers are asked to be satisfied with locks that will chamber two boats; whereas, a canal that would pass the 25 000 000 tons naturally seeking interchange between the waters of the Hudson and Lake Erie would chamber 20 boats of the same size. At the Sixth International Inland Navigation Congress Mr. C. H. Stieltjes showed that boats of 6.5 ft. draft were allowed a speed of 9.3 miles per hour in the ship canals of Holland; and American shippers are asked to be satisfied with a possible maximum speed of 4 miles per hour.

The establishment and maintenance of such great frictional resistance to the interchange of commodities between the producing and consuming centers of this country is advocated, in part at least, by many in this State, for the sake of the charges that can be levied on the passing traffic, to judge from a sentence in the last report of the State Superintendent of Canals, which reads:

"It is clearly evident that the duty of the Legislature will be to provide the means for the construction of the largest possible canal, intended for the navigation of boats of such construction as will be able to navigate the inland waters of this State, discharging and receiving their cargoes at Buffalo and New York, or other intermediate points."

If that does not imply a continued burden on the production and consumption of the United States in favor of a few, the words quoted are misleading. The spirit that dictated that sentence recalls the early part of the seventeenth century, when Henry IV found it necessary to forbid the clergy of France tolling the water-borne provisions *en route* to Paris. It is further opposed to any adequate conception of the true function of transportation. Mr. North.

The easily recognizable beneficence of transportation through ample channels, and with free competition, is a lower charge for the service; but as all costs for transportation must be taken in some ratio from the gains of both producer and consumer, any lowering of the cost of freightage is a direct contribution to the wealth of the community in which it occurs and enables that community to both draw its supplies and exchange its commodities over a larger area. It is the low, or comparatively low, cost of assembling the raw materials for manufacture or consumption, and distributing the finished products, that governs the locations of the great centers of manufacture and consumption.

Industrial history shows this to have been true all over the world. Holland easily took the primacy in manufacturing from the Italian cities while they were still financial centers through its use of the many mouths of the Rhine and the Maas, where they scatter themselves through the low country, and it held that primacy until England commenced improving its water-courses, and for some time afterward. This fact has been greatly obscured by the success of the British Navigation Act, passed in 1651. That act, doubtless aided by the attacks of Louis XIV, broke the maritime power of the Dutch within 50 years, and, "under God's good providence," has done more to extend the wealth and influence of the English-speaking people than all other legislative enactments.

But the English commenced the improvement of their roads as early as the reign of Henry VIII, when an act was passed for the better government of the religious houses and the mending of the highways. In the time of Elizabeth the first canal act was passed, but the great flood of canal bills did not come in until 1758, when Brindley commenced building canals. These canals immediately reduced the freight rates in England, more particularly in the manufacturing counties, to from one-third to one-fourth of what they had been previously, and they made England the manufacturing country that it has been since.

As pointed out by a prominent economist, it was the power to manufacture that enabled England to subsidize her allies on the Continent during the Napoleonic wars, because the gold they received from Britain came back to that country for manufactured goods. During the whole period of these wars, England increased in wealth. A like

Mr. North. effect was seen in the Dutch war for independence, as Philip had no other place to go for manufactured goods and munitions of war than Holland, and the very exertions which he made to conquer the Dutch added to their wealth, so that at the close of the war, Spain, although it drew on the wealth of America and the Indies, was impoverished, while the Dutch were rich. It was Brindley, with his coadjutors and successors, who sent Napoleon to Elba and Helena, rather than British soldiers and statesmen.

In 1830, when England left off canal building and commenced her railway construction, she had more miles of canal and improved waterways per given area than the United States now has of railroads. Her very rapid railway building up to 1847 confirmed her position as a manufacturing nation, and it was thought for years that that position was unassailable. A great American authority on iron manufacture said in 1870 that Great Britain was making more than half the iron of the world, and she would probably continue to do so. As a matter of fact, in 1870, the total production of iron was about 11 000 000 tons, and England made 54% of this; the United States made 15%, and Germany made 10.5 per cent. But in 1890 the production of the world was about 25 000 000 tons; the United States made 37%, Great Britain made 31%, and Germany made 18 per cent. He did not comprehend the influence of this then rapidly diminishing cost of transportation in the United States.

The rate of transportation in Great Britain for nearly fifty years has not been noticeably reduced. It is virtually as high now as it was in 1846. There is not in Great Britain a canal or a water course that is not in the hands of a railway company, a trust, or a private corporation; tolls are charged on every one, and very few have been enlarged or improved. It now costs about as much to get a bill through Parliament for a mile of railway as it costs in this country to build a mile of railroad. In the meantime, other countries have increased the size of their waterways, have made them free, and effected noticeable reductions in freight rates. Up to about 1870 the British had the advantage of the lowest internal freight rates; they held their primacy in manufacturing and controlled the price of finished products. To-day, no manufacturers pay as high a rate per ton-mile for assembling raw materials and distributing finished products as the British, and Americans pay the lowest rate. In spite of the former's ownership of between 60 and 70% of the world's effective ocean tonnage, they have given up the first place as manufacturers to the United States, and are now fighting with Germany, which is enlarging its waterways and building new canals, to retain the second place.

Apparently no English engineer has felt called on to say publicly what some have said to the speaker. If Great Britain is to continue to approximate to even its old position as the chief servant of the world,

it must nationalize, free and enlarge its canals. This, however, is held to be impossible while the power of capital invested in railways retains its influence in Parliament. Mr. North.

Mulhall puts the average railway freight rate in Great Britain at 2.8 cents per ton-mile. This, it should be noted, is a matter of estimate, as the railway managers of that country prefer not to make returns of their ton-mileage. There has been no appreciable change in British freight rates for nearly fifty years, for although a parliamentary commission lately reduced the posted rates, the classification was altered so that in some instances the rates were increased. In this country the average freight rate in 1865 was probably very near 4 cents per ton-mile, and now it is about 9 mills, or less than one-fourth of what it was.

Neither the ton-mileage nor the average freight rates for the whole country was ascertained until 1882. For that year and for subsequent years both have appeared in "Poor's Manual." They commence with 39 302 209 249 ton-miles carried at an average charge of 1.236 cents, and end with 82 289 400 498 ton-miles carried at an average charge of 0.851 cent (see Fig. 1). From 1865, however, "Poor's Manual" for some time has compiled the ton-mileages and rates of two groups of railroads, the Boston and Albany, the New York Central and Hudson River, the Erie, the Pennsylvania Central, the Pittsburg, Fort Wayne and Western, the Lake Shore, and the Michigan Central, designated on the diagram as the seven trunk lines; also the Illinois Central, the Chicago and Alton, the Chicago, Rock Island and Pacific, the Chicago, Burlington and Quincy, the Chicago and Northwestern, and the Chicago, Milwaukee and St. Paul, designated as the six western roads. The same diagram also contains the freight rates charged on all Massachusetts roads for 1871 and subsequent years, with the ton-mileage from 1872. These roads cover mileage outside of the commonwealth which is owned or leased by and forms parts of Massachusetts roads; the lines radiate into nearly all New England.

It will be noticed that in the case of both the trunk line and western groups the freight rates have been decidedly lower, and have fallen faster, than on the Massachusetts roads. Although, as "Poor's Manual" groups the roads of the country, the New England roads have 40% more traffic per mile than the average, the freight charge is 45% higher than the average. Massachusetts and New England education generally seems based on English ideas. It is said to be harder to get a charter for an opposition road in that section than in any other. Since about 1880, when the divergence in freight rates becomes noticeable, heavy manufacturing has been deserting Massachusetts and the country served by its roads. It is thought that lighter manufacturing may follow. Concurrently a demand for free raw material has risen. It is probable that free railroad building, with the consequent cheaper freights, would serve both that locality and the whole country better.

Mr. North. Accompanying this general fall in railroad rates and increase in traffic, there has been a wonderful development of the Lake commerce. The members of the Society are probably all aware of what Albert Fink, Past President Am. Soc. C. E., and Mr. G. R. Blanchard have said about the influence of water rates on railroad rates, and though some people hold that the railroad wars have been the compelling force in the reduction of water rates, there can be no doubt that as the rates by rail and water have decreased, the traffic has increased. This is shown in Fig. 2, where the traffic through the St. Mary's Falls Canal, leading out of Lake Superior, is the prominent feature. The traffic on the Erie Canal from the opening of the St. Mary's Falls Canal is also given, to which is added, as a matter of comparison, the traffic on the Suez Canal.

It will be immediately noticed that as an appliance for the service of the general public the Erie Canal, like the industrial position of Great Britain, is a decaying one. In 1880 the traffic was at its maximum in amount, and in 1894 it had fallen to 68% of that volume. For the ten years ending with 1869 the tolls collected by the State of New York on merchandise passing through canals averaged over $10 000 000 per year. This was possible when the freight trains on the New York Central Railroad averaged 25 to 30 cars carrying 8 tons each; but a canal that will only pass boats of from 250 to 300 tons burden has no excuse for existence when competing with locomotives that will haul over 1 000 tons from Buffalo to New York. Hence, even after all tolls were abolished, the value of the canal as a channel of distribution has been decreasing.

The Suez Canal, which was opened in the fall of 1869 in the service of a trade not unknown in the time of Solomon, led to a great development of traffic between Europe and Northern Africa on the one hand, and Asia, Australasia and Eastern Africa on the other, but it may have reached its maximum usefulness, and last year its traffic was only about 60% of that through the St. Mary's Falls Canal, built in the service of what was still a wilderness 25 years ago. The apparent arrested development of the Suez traffic is not due to the competition of cheaper routes (freighters through that canal claim not to receive as many shillings per ton of cargo now as they received pounds sterling when it was opened), but communities working for Asiatic wages do not seem able to acquire those wants which are necessary to the development of a large freight movement.

The St. Mary's Falls Canal was opened in 1855, with the lift divided by two locks with 11½ ft. on the mitre-sills. In 1881 a single-lift lock, with 16 ft. on the mitre-sills, was opened and the canal freed from tolls. Three years after, the draft of 89% of the traffic passing was greater than the depth of the old locks. Up to 1881 the development was fairly satisfactory; since then it has been phenomenal. Although the

Mr. North.

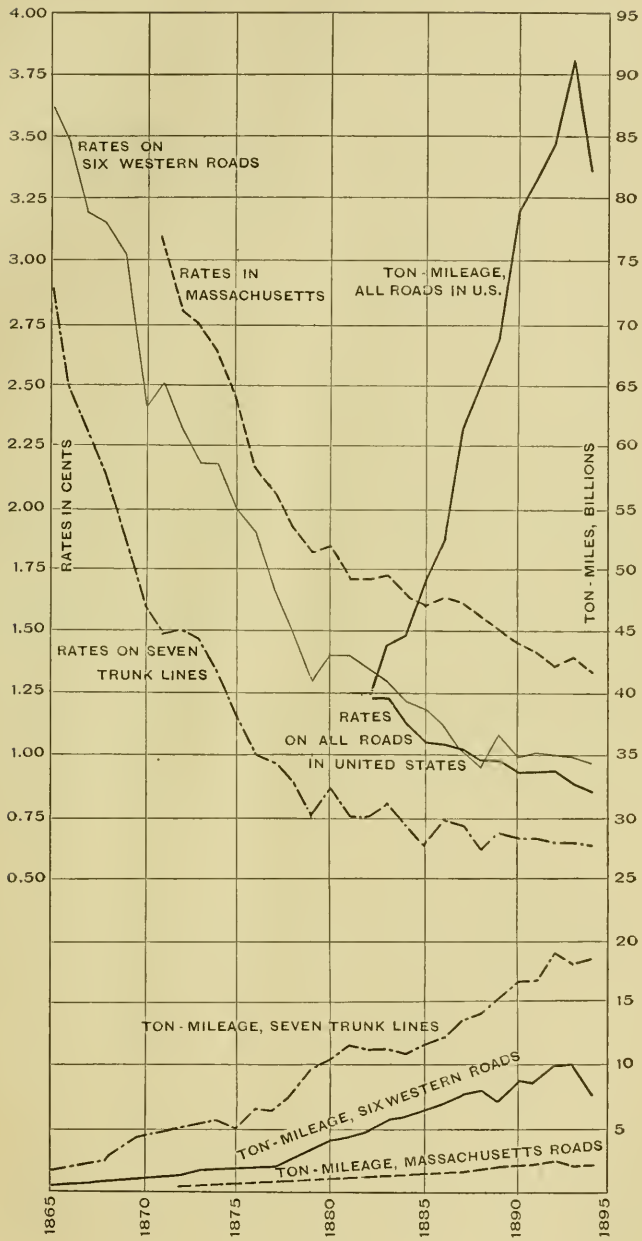

FIG. 1.

Mr. North. aggregate mileage of the Lakes, from the heads of Lakes Superior and Michigan to Buffalo, cannot be counted as more than 1 400 miles, and there are over 178 000 miles of railroads in this country, last year the ton-mileage through that canal was equal to 12.6% of the ton-mileage on all the railroads of the United States, and for some time the ton-mileage on the Lakes above Buffalo has been equal to about one-fourth of the railroad tonnage. Incidentally the populations of Buffalo, Cleveland, Detroit and Chicago have increased during twenty years at the rate of 209%, while for the same time at Cincinnati, Louisville, St. Louis and New Orleans, large cities in the same longitude but on water-courses which have not been materially improved or deepened, the increase has been at the rate of 40.2 per cent.

Writing in 1891 in favor of a 20-ft. lake channel, the late General O. M. Poe, M. Am. Soc. C. E., whose successful exertions for deep lake waterways have added so much to the wealth of this country, said: "During the season of 1890 over 9 000 000 tons of freight passed through the St. Mary's River, and more than 22 000 000 tons through the waterway between Lakes Huron and Erie. The increase in the navigable depth of channels on the Lakes from 9½ ft. in 1852 to 16 ft. in 1882 developed this commerce, and it is only reasonable to expect that a further increase of 4 ft. will be followed by a corresponding increase in shipping."

Continuing he says: "For nearly 30 years I have watched its increase, but neither I nor any one else within my knowledge has been able to expand at the same rate. The wildest expectations of one year seem absurdly tame the next."

This year it is claimed the wonderful figures of last year will be increased to 16 000 000 tons, an increase rendered possible by the commercial confidence in the future of lake freightage born of work on the 20-ft. channels. But any year's wild expectation of an increased traffic that must be strained through the proposed apertures in the enlarged Erie Canal, 8 x 18 ft. in cross-section, will seem hysterical the next.

The "Report on the Internal Commerce of the United States for the year 1891," contains an estimate that the saving to the public by the water transportation of the traffic on the Lakes for 1890 was $135 000 000, comparing lake freight charges with the railroad rates, which the competition on the Lakes have reduced. This is equivalent to a dividend of 66% on the total expenditures by the nation for river and harbor improvements. This was for some 9 000 000 tons carried at an average rate of 1.3 mills per ton-mile. Last year there were 13 000 000 tons carried for 1 mill per ton-mile, and 33 000 000 tons passed Detroit. Although in the April, 1891, number of the *Engineering Magazine*, and subsequently, figures of like import have been frequently presented to the public in technical journals and Government publications, persons are found who will gravely make and publish computations showing how much toll should be charged on a ship canal through this State.

Mr. North.

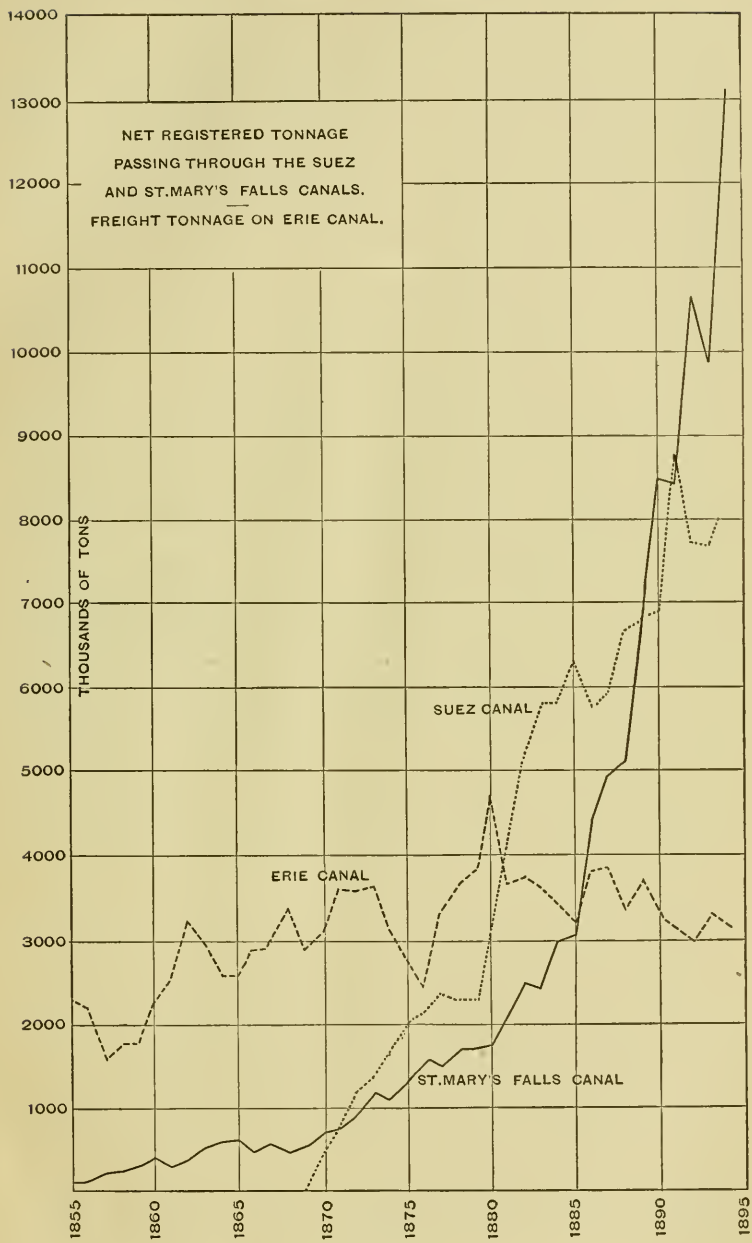

FIG. 2.

Mr. North. But the value of the deep waterway on the Lakes is not by any means confined to the money saved in freighting through their channels. The greatest good is derived from the industries they stimulate. E. S. Wheeler, M. Am. Soc. C. E., the engineer in direct charge of the canal at the foot of Lake Superior, estimates that fully half the freight carried would not sell at lake docks where it is landed for the cost of railroad freight from its point of origin. So that without the enlarged waterway the country would have lost the profits and employment engaged in the production of some 8 000 000 or 9 000 000 tons of merchandise.

Taking the item of iron ore, for instance, its value to the country is not limited by the $18 000 000 or $20 000 000 it is sold for on the lake docks, but by something in the neighborhood of $175 000 000, the probable value of the finished product. In addition to the interest in the securities of railroads built up and maintained by the transportation of this material and the supplies of those who manufacture it, all who are engaged in its production, or have occasion to use the finished product or any other thing the production and marketing of which is rendered profitable by the cheap freights on the Lakes, all are beneficiaries of those enlarged waterways.

Two other diagrams are shown in illustration of the effect of deepened and enlarged waterways. Fig. 3 shows the tonnage of ocean and inland vessels at the port of Montreal, with the increased depths obtained between that port and the sea. Fig. 3 also shows the railroad and river tonnage at Frankfort-on-the-Main. The traffic on this river, which had a governing depth of 3 ft., fell through the competition of railroads to about 10 000 tons, when it was decided to increase its depth to 8¼ ft. The increased traffic in both of these cases that has followed deeper channels has been marked. It is hoped that these diagrams will, without further comment, show a direct relation between enlarged channels of interchange and decreased freight charges on the one hand, and increased production and distribution on the other.

A great deal of the opposition to improved waterways, not only here but in other countries, is through a fear that the value of money invested in railroads will be injured, the entire fall in freight rates being sometimes counted as lost income. If that reasoning were correct, the loss to railroad investors on the business of 1893 would be nearly $2 700 000 000 as compared with the prices of 1865. The net results to the people of this country have been that either supplies may be drawn and customers reached over an area with a radius four times as great, or for an equal area three-fourths of the freight money can be divided between the producer and consumer.

Probably the increased ton-mileage, which is thought to be greater in this country than in any other, is due partly to the augmented cir-

culation through the larger area, and partly to the saving in cost which creates a fund that is available for increased consumption. But it is doubtful if the railroads have lost much by the decrease in freight rates. Some investigations made in connection with a paper read at Mr. North.

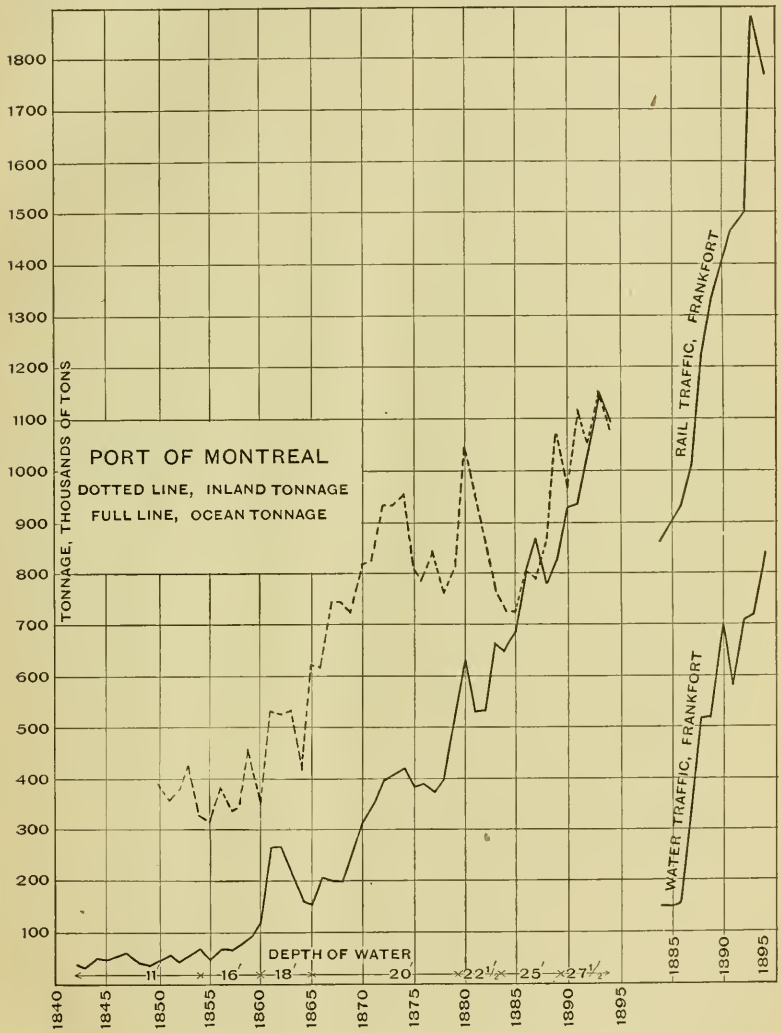

FIG. 3.

the Fifth (Paris) International Internal Navigation Congress showed that in the case of the seven trunk lines above referred to, the stocks were virtually at par on both July 1st, 1865, and 1890; that during the first-mentioned year, with a freight rate of 2.9 cents, the net income

Mr. North. was 29.207%, and for the last-mentioned year, with a freight rate of 0.68 cent, the net income was 31.95 per cent. Ten times as much traffic was handled at one-fourth of the cost per unit, with greater profit.

There are three propositions before the country : That the Erie Canal should be enlarged to a depth of 8 ft. on the mitre-sills ; that it should be made a ship canal ; that a ship canal should be built through Canadian territory and the produce of the West be forced to seek a market through the mouth of the St. Lawrence. The proposition in the paper to build a ship canal into Lake Ontario, so as to split the traffic with Canada, and prevent the population between Buffalo and Syracuse growing as that on the lake shore does, seems a modification of the last-mentioned plan.

The St. Lawrence plan seems attractive to certain minds. At an International Deep Waterways Convention held at Toronto last year, Mr. Smalley, a delegate, proclaimed : "It is our duty in equity, as well as in self-interest, to contribute towards deepening the channel down the St. Lawrence," and, claiming to speak for a large portion of the West, he said : "You will not find among the delegates from the Northwest any national prejudice that would stand in the way of our joining you in what is unquestionably the best and quickest way to get to the sea, because these two great English-speaking peoples," etc.—a formula generally used when any material interest of the United States is to be permanently injured.

There are many self-called broad-minded thinkers among us, without "any national prejudice" when looking at the interest of their own country, but strenuous vociferators for the equities and interests of other lands. The type is not unknown in the engineering profession; a member of it has said: "This is an engineering question in its broadest sense, and we should approach it, not as New Yorkers or even as Americans. A narrow patriotism may stop at the borders of our own country, but professionally," etc.

Let us examine the position of an engineer whose professional advice should govern the investment of money for the service and development of larger or smaller areas.

All know that from Chicago or Duluth to either New York or Montreal, it is substantially 1 500 miles. But from Montreal to the sea it is 1 000 miles, and from Thanksgiving Day to the first of May that 1 000 miles is ice-locked, so that any merchandise not cleared from the port of Montreal before or by December 1st must pay storage for five months, nor can any return goods be delivered during that period. At the port of New York, on the contrary, imports have free access and exports have free egress every day of the year, and storage is unnecessary.

Neglecting the broad and extrinsic patriotism of one who volunteers to turn out of his own country the many advantages accompany-

ing a great tide of commerce, it is submitted that professional advice to build the great avenue for the interchange of products between the interior of America and the sea coasts of the world to a five-months' wall of ice is not, on the whole, proposing to direct the great forces of nature to the service of mankind. Mr. North.

There is another consideration, not so patent as the ice in the St Lawrence, but more weighty. The main thoroughfare of any country should intervene between the great body of producers and their largest and best market. That for the American farmer is New York. That for the New Yorker is offered by the American farmer. In the census year 1890 the value of products manufactured in the city of New York was $777 222 721, and the value of goods manufactured in the six cities on the waters of New York Bay, viz., New York, Brooklyn, Long Island City, Hoboken, Jersey City and Newark, was $1 200 671 459. In the same fiscal year the total value of merchandise imported into the United States was $789 310 409. If all of the best authorities on political economy are not mistaken in their assertion that "commodities exchange for commodities," the markets of the world outside of the United States are worth 3% more than the market of the city of New York, and the consumption of the six cities bordering on the Bay of New York is worth 50% more to American farmers than the consumption of all foreign countries. The wages paid in manufacturing industries in the six cities above mentioned are equivalent to almost 42½% of the total value of the country's imports. These relations are shown in Fig. 4.

On the other hand, Montreal is an inconsiderable place of 216 000 inhabitants, with a total city revenue of nearly 80% of what the city of New York expends for sweeping its streets, and of about the same manufacturing importance as Schenectady. It is doubtful if all of Canada offers as good a market to our farmers as some of the numerous cities and towns on the line of the Erie Canal, and there are actually men who are led to believe that they are reaching for markets when they try to send the produce of the Northwest to sea *via* Montreal.

It is generally admitted that the cheaper rates of a large waterway, by stimulating the bordering industries, augment the traffic and profits of parallel railroads. The converse is also true. Large manufacturing industries require the services of railroads for the quick delivery of finished products and the necessary circulation of men engaged in such industries. Paralleling the line of the Erie Canal and jointly dependent on the industries and wealth built up by it for returns on the investment, one company has expended over $160 000 000; while in the part of this State which it is proposed to sidetrack by a canal into Lake Ontario, the line of the canal is crossed by competing roads every few miles. These adjuncts to the cheap transportation which an enlarged canal will offer make this part of the State an

Mr. North. ideal location for manufactories of all kinds, but the route by the St. Lawrence is virtually without such aids to industrial development.

It seemed to the speaker that the most advantageous route for a great ship canal connecting the commerce of the Lakes with that of the Atlantic, and serving, not only the largest number of consumers, but, to use the words of James Bryce, Esq., consumers "who allow themselves luxuries such as the masses enjoy in no other country," is substantially on the line laid down by Elnathan Sweet, M. Am. Soc. C. E., in his paper of June 10th, 1894, "The Radical Enlargement of the Artificial Waterway Between the Lakes and the Hudson River."

This line, passing as it must near the outlets of Cayuga and Seneca Lakes, would connect nearly 80 miles of deep-water navigation to the canal, and make Ithaca and Watkins, the towns at the heads of

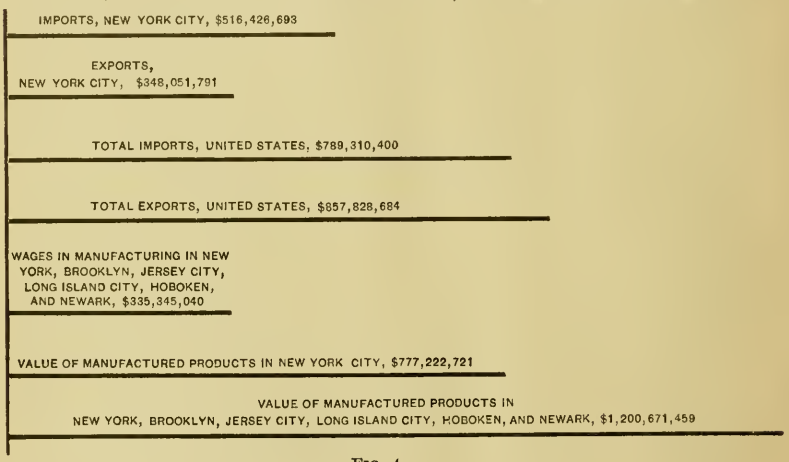

FIG. 4.

these lakes, commercial ports of much importance, which, by the aid of a ship canal to Oswego, would dominate most of the trade of Lake Ontario and the upper St. Lawrence, instead of making the trade of all that region tributary to Montreal.

Reference to the causes which have made the city of New York the largest manufacturing city in the world, and incidentally draw 65% of the imports of the United States to this port, may not be out of place. It will be noticed that the position of the city is unique. It is situated at the mouth of a stream so commodious and grand that the *Great Eastern* could have ascended it for 120 miles, instead of at the head of navigation. The governing factor in New York's position and wealth is Long Island Sound. In addition to this, there is another navigable channel to the westward, the Kill von Kull. The report of the Chief of the United States Engineers for 1884 puts the tonnage con-

tributed to the harbor of New York through this channel at 16 500 000 tons. A later report says the tonnage of this channel is greater than that through the Narrows. Colonel W. M. Gillespie ascertained that the tonnage of the Hudson River was 18 500 000, of which 3 500 000 tons is contributed by the Erie and Champlain Canals. Mr. North.

When to the happy meeting of these natural channels of large capacity the tribute of the Erie Canal was added, there was no other place where the raw materials for manufacture could be assembled so cheaply, or from which the finished products could be more easily distributed. Hence all the railroads of the country pointed to or connected with it, and it became the seat of the largest production and consumption. Unless it becomes desirable to remodel the railroad system of this continent and reincur the expense necessary to the accommodation of nearly 3 000 000 people, the interests already centered in and about the city of New York demand such an ample waterway as shall now and in the future control the direction of commerce and augment the wealth and prosperity of this country, rather than that of a country which some hope to possess in the future.

Chauncey N. Dutton, Esq.—In discussing the project of a ship canal from the Great Lakes to the seaboard, the speaker did not propose to limit the range of thought to the narrow field, the grain carried by the Erie Canal. He stated that more than 9 000 000 tons of vegetable food are carried from the Lakes to the sea. The Erie Canal does not carry one-tenth of it. From 70 000 000 to 75 000 000 tons of through freight are carried on the routes paralleling the future continental highway, while the Erie Canal carries about one-thirtieth of it. The ship canal must be powerful enough, as a means of transportation, to compete with those carriers which have proved stronger than the Erie Canal. It will not be a competitor with the State canals; it will be a feeder to them, filling them up with local distributing business, which they can do cheaply and profitably. It may relieve them of a part of the through business, for which they are ill adapted, because it can do it cheaper, as can be proved by the author's own figures, and also by reliable data. Mr. Dutton.

The estimate in the paper, that in a vessel carrying 5 000 tons on 16 ft. the cost of transportation will be 5 cents per bushel, the speaker considered too high, in view of the present cost of conducting transportation in cargoes of 2 000 to 2 500 tons.

The estimate that in a vessel carrying 7 000 tons on 20 ft. draft the cost will be 3.22 cents per bushel, the speaker said was 2.5 times too high, as shown by an analysis of the author's figures, corrected in some particulars to agree with the present performance of steamships on the Lakes.

With regard to the third and fourth estimates, the speaker said that it is proposed to fit the Erie Canal locks with improved gates,

Mr. Dutton. which will add 12% to the tonnage of the barges using them. The draft in the locks is to be 8 ft., instead of 7.5 ft., and it can be deduced from the author's second estimate that wheat can be carried from Chicago to Buffalo for 0.75 cent instead of 1.25 cents, as he states. By making these corrections, he could have made a showing for the Erie Canal some 20% more favorable than he has. As a compensating error, however, he has assumed that barges can be towed at an average speed of 4 miles an hour in the Erie Canal.

The hypothetical barge, 180 ft. long, 17.2 ft. wide, 7.5 ft. draft and presumably about 11 ft. deep, could not have the same weight, cost, carrying capacity and expenses per foot of length as the real 90-ft. barge of equal width and draft, but her weight and cost would increase nearly as the square of her length, and, therefore, the hypothetical 180-ft. barge would weigh and cost nearly four times as much as the real 90-ft. barge, because, assuming the wave action in a seaway to be equal upon both, the weight would be nearly as the square of the length, as in beams. The 90-ft. barge weighs about 75 tons.

To settle this matter beyond controversy, the speaker obtained an approximate estimate of the weight, cost and carrying capacity of such a barge from one of the largest and most successful shipbuilding concerns in the country. Its chief engineer states that, "roughly speaking, the weight would be about 280 tons; the cost, $22 000, complete, ready for sea, and the carrying capacity about 360 tons. Such a barge would be of no service whatever if it were built." Such a barge never has been built, and if it were built it could not be towed singly around the short curves in the Erie Canal at the rate of 4 miles an hour; in fleets, not at all. Singly it might possibly be towed at the rate of 1 mile in 4 hours, by two tugs, one to pull her nose off and the other to pull her stern off at every curve.

That such a barge is not available for the lake traffic he believed clearly proved by the history of navigation on the Canadian canals. For nearly a generation there has been an open waterway with 9 ft. least draft. and only 54 locks, the smallest 45 × 220 ft., big enough for two of the hypothetical barges to lie in, and used by barges of much more advantageous proportions than the author assumes. The St. Lawrence barges never engaged in trade across Lake Ontario and through the Welland Canal. They are shut out of the lake traffic by steamers of 1 650 tons capacity, less than one-fourth of the size of the *Victory*, by vessels which cannot earn a living west of Port Colborne.

The tenacious hold of the barge idea is surprising in view of the facts. Had there been any such economies in barges as have been claimed, every shipyard on the Lakes would be building them; but, on the contrary, the one yard dedicated to barge construction is now building as many ships as barges.

The author gives the operating expenses of the *Victory* as $250 per

day, and her interest and other fixed expenses as $67 500 per year. The per diem ratio of these expenses he makes 1 to 210, basing it on a short season of lake navigation. He forgets that when the *Victory* can get to the sea, she will not go back to winter in the lakes. She will put in surface condensers, and engage in the coasting or foreign trade during the winter months, and the per diem ratio of her fixed expenses will shrink from 1 to 210 to 1 to 365, or from $321.43 to $184.93 per day, her total daily cost shrinking from $571.43 to $434.93, as the immediate result of making the ocean accessible to her. This alone reduces his estimate 24%, or from 3.22 cents to 2.45 cents per bushel. He figures on a route from Chicago to New York, 1 363 miles long, with 350 miles of canal and 50 locks—the Erie Canal route. A much better route will be *via* the St. Lawrence and Lake Champlain, 1 570 miles long from Chicago to New York, with at first only 90 miles of canal, and finally 75 miles, and only five locks. Mr. Dutton.

Applying his basis of speed to this route, 1 480 miles of open water at 13 miles per hour equals 113.85 hours, 90 miles of canal at 7 miles per hour equals 12.85 hours, and five locks at 24 minutes per lock equals 2 hours, a total of 128.7 hours, or 5 days 8 hours 42 minutes steaming time from Chicago to New York. The author estimates the time *via* the Erie route at 153 hours, 24 hours 18 minutes longer than the St. Lawrence route, or longer in the ratio of 100 to 24. Here is a further reduction of 16% in one item, the actual steaming time.

The author bases his estimate on the supposition that the ship would be tied up in port 13 days each trip. As a matter of fact, under favorable circumstances at Chicago or Duluth, she would discharge and receive her cargoes and clear the port within 12 hours of the time she entered. In the pamphlet, "The Great Lakes of North America," by the late George H. Ely, Assoc. Am. Soc. C. E., it will be seen that the ship *Manola* was in port an average of only 37 hours 36 minutes per trip during the entire season, her trips averaging 1 686 miles. This is on the Lakes. The author's assumption therefore means that when in New York the ship would tie up 11.5 or 12.5 days, as against 0.5 to 1.5 days in the lake ports. It was a violent wrench to the speaker's business instinct to assume any such thing. If New York does not provide good terminal facilities, other people will come and build them, as they built the elevated and cable roads, and many other things. But facilities for rapid loading and unloading now exist in New York for such cargoes as are in sufficient quantities to warrant them. It is a weekly occurrence that great ocean steamships are entered and cleared within three days; and it has been done within two days.

But suppose the terminal facilities not to exist, the speaker asked would a ship tie up here 11.5 or 12.5 days, when in that time she could carry her cargo to Europe, earning 4.75 to 6 cents a bushel; or could

Mr. Dutton. coast from Halifax to Galveston, or make a round trip to the West Indies and back, or run light to Chicago and Duluth and bring back another cargo. He believed that what the ship will actually do, on the author's basis of speed and cost, is this: She will steam 1 480 miles in open water in 113.85 hours; 90 miles of canal in 12.85 hours, and pass five locks in 2 hours. a total of 128.7 hours ; and will lose at the outside 37.6 hours in loading and unloading, a total per trip of 166.3 hours.

The author's basis makes her cost $434.93 per day, equal to $18.12 per hour, and the cost per trip of 166.3 hours between Chicago and New York would be $3 013.36, equal to 43 cents per ton, or to 1.29 cents per bushel, only a trifle more than the cost of breaking bulk at Buffalo, and 40% of the estimate in the paper. This is why the western shippers want to ship through and avoid breaking bulk, and why they demand ship canals. It is because it will save them 3.11 cents per bushel, or $1.161 per long ton over the cost of transportation in steel barges steam-towed *via* a future 9-ft. Erie Canal, according to the author's fourth estimate. This is on the supposition that the ship finds it more profitable to discharge at New York than to carry her cargo elsewhere, and finds a return cargo, which might be anthracite coal, sugar, building stone, bricks, ore, lumber, asphalt or package freight. If she found no cargo at New York, she could run light to Kingston for anthracite, or to Fishkill, Troy or Burlington for building stone, marble or package freight from New England; or to Lake Champlain for ore, or to Quebec for fish or iron sand, or to Oswego for anthracite, or to Buffalo or Cleveland for bituminous coal or coke. At the very worst, suppose she had to return light every other trip, she could run light to Chicago or Duluth for east-bound freight, in which case her account would stand as follows: East-bound trip and detentions, 166.3 hours; one-half of west-bound trip, 64.35 hours, a total of 230.65 hours, at $18.12 per hour, equal to $4 179 38, equivalent to 59.7 cents per ton, or to 1.6 cents per bushel, less than half of the author's estimate. This is the most unfavorable showing that can be made. New York is one of the three first-class ports of the American continent, and can provide return cargoes nine trips out of ten. Even were New York unable ever to provide a return cargo, and the port sealed so that the ship could not get out to find cargoes elsewhere, and were there no intermediate ports, and the east-bound freight had to bear the expenses of the round trip, the cost would only be 76.4 cents per ton, equal to 2.05 cents per bushel, less than two-thirds the author's estimate, and less than half the cost in steel barges *via* a 9-ft. Erie Canal.

The actual cost of operating ships of various sizes and of conducting transportation in them under various conditions was taken up by the speaker, who said that for the figures presented he was indebted

to the courtesy of correspondents familiar with the subject. As such statements might seriously affect important business interests, he merely presented conclusions in a summarized form. Mr. Dutton.

Attention was called to the published balance sheet of the Minnesota Steamship Company's ship *Manola* for the season 1890, which, so far as he knew, is the only authoritative statement of the kind ever published. The *Manola* is 292 ft. 5.5 ins. on the keel, 308 ft. 5.5 ins. over all, 40 ft. breadth, 24 ft. 6 ins. molded depth, 2 000 H. P., and can carry, fully laden, about 3 500 tons. In 1890 she was in commission 222 days, in port 47 days, steamed 175 days, made 30 trips of 1 686 miles each, equal to 50 584 miles or 227.5 miles per day, her speed being 11.85 miles laden, and 12.72 miles light, the average being 12.25 miles per hour. Her fuel consumption was 5 528 tons, worth $14 427.14. She burned per hour 209 lbs. when light, 226 lbs. when loaded, and averaged 218 lbs., equal to 1.5 ozs. per ton-mile. She carried 71 170.69 tons, equal to 2 295.816 tons per trip on 14.7 ft. to 14.9 ft. of water. She loaded in 7.5 hours, and unloaded in 12.5 hours. She earned $93 738.25 gross, $38 624.05 net; her expenses being $55 114.20, equal to 58.79% of her gross earnings.

Carrying 2 295.816 tons, less than one-third the capacity of the ship under consideration, she cost $248.26 per day, including insurance but not amortization, and earned $173.98 net per day, out of which she paid amortization and a very handsome profit to her owners. It will be observed that she cost daily only $1.74 less than the author's estimate of the running expenses of the *Victory;* and that the ratio of operating expenses to amortization and profits was as 100 to 70, and she was thought to do nobly. The author's assumed ratio is 100 to 129, a difference of 59 per cent.

The speaker next gave a summary statement of the actual cost of conducting transportation during the current season, up to October 1st, in a large fleet of lake steamships, which are a fair average of the large lake freighters. In the statement furnished to him, some of the calculations were based on gross tons and some on net tons, but all have been reduced to gross tons for better comparison with the statement of the *Manola.*

GREEN BAY PORTS TO LAKE ERIE PORTS.

Water.		Average cargo. Gross tons.		Cents.
15 ft. 4 ins.	Ore and return light.....	2 515.43		44.09
15 ft. 4 ins.	Ore down	2 546.36		36.28
	Coal up.....	1 869.54		29.56
	Average cargo.....	2 207.93	Average.	33.43

Mr. Dutton.

DULUTH TO LAKE ERIE PORTS.

Water.		Average cargo. Gross tons.		Cents.
14 ft. 2 ins.	Ore and return light.....	2 136.73		64.51
14 ft. 2 ins.	Ore down	2 009.17		44.45
	Coal back	1 934.21		35.08
	Average cargo.....	1 971.69	Average.	39.86

To the above must be added 4 cents per ton for painting, winter laying up and fitting out expenses, which would make the cost between Duluth and Lake Erie ports as follows on 14 ft. 2 ins. of water:

Vessel returning light, average cargo,	2 136.73	gross tons,	68.51	cents.
" carrying ore down, " "	2 009.17	" "	48.85	"
" " coal back, " "	1 934.21	" "	39.08	"
Average cargo	1 971.69	Cost...	43.86	"

Average cost of carrying 1 ton per round trip, 87.72 cents.

The statement of the *Manola* gives the cost per ton of freight carried an average of 1 686 miles as 77.4 cents, the draft being 14 ft. 7 ins. to 14 ft. 9 ins., and the cargo 2 296 tons. The vessels in this fleet average within 1 ft. of the length of the *Manola*, and are 3 ins. wider. The comparison of the figures, 77.4 cents per ton per trip by the *Manola*, and 87.72 cents per round trip by this fleet, is not fair to the fleet, because from the length of the *Manola's* voyage it is evident that she touched at intermediate ports, probably loading down as deep as she could in Lake Superior, and then running to Green Bay to complete her cargo. The speaker changed the figures for the fleet to make comparisons with the *Manola* more just. It cost the *Manola* 22.6 cents per ton to handle cargo. At the present time the charge for unloading is 15 cents per ton, and for trimming 2.5 cents, making a total of 17.5 cents.

If from the figures for the fleet 17½ cents are taken, and from the remainder 10.5% is deducted to compensate for being twice in port and loaded and unloaded as against once for the *Manola*, results are obtained which admit of an approximately correct comparison. This gives a cost of 62.85 cents per ton for a trip equal to twice the distance from Cleveland to Duluth, which would be approximately equal to the length of the *Manola's* trips. The average size of the fleet steamer is practically the same as that of the *Manola*, but the *Manola* had the advantage of 5 to 7 ins. of water, which enabled her to carry 325 tons additional cargo. Again correcting the figures for the fleet steamer, with this additional draft of water, she could have carried a cargo for 56.44 cents instead of 62.85 cents, the lack of from 5 to 7 ins. of water adding

6.41 cents to the cost of carrying cargoes a distance approximately 1 650 miles. Mr. Dutton.

The vessel's performance on this voyage may, without considerable error, be assumed as equal to her performance on a voyage from Duluth to New York, because if the detentions between Cleveland and Duluth be equated with those between Cleveland and New York, the equated distances, east and west, will be substantially equal. Therefore, the cost of conducting transportation between Duluth and New York in a vessel carrying 2 300 tons on 15 ft. of water would be 56.44 cents per long ton, or 1.51 cents per bushel. The fleet steamer's round trip performance, without corrections as above, is 87.72 cents per long ton, or 2.35 cents per bushel. To these must be added interest and amortization. The net earnings of the *Manola* per season of 222 days were 70% of her operating expenses. Had she been in commission 365 days and earned the same sum of money, the percentage would have been 42⅝. Adding this percentage to the figures given above, there results 2.15 cents corrected rate and 3.83 cents uncorrected for the remunerative freight rate in a vessel carrying 2 300 long tons on 15 ft. of water.

Through the courtesy of the executive officer of a great shipyard, the speaker was able to furnish some further light upon the effect of depth. The following are the data with reference to a ship which is to be 432 ft. over all, 412 ft. on the keel, 48 ft. beam and 28 ft. depth. It is to be a very heavily constructed, staunchly built hull, good not only for the present, but, if kept off the rocks and out of collisions, good for several generations.

Dead Weight Capacity in Net Tons, Including 150 Tons of Fuel.

	Net tons.		Net tons.
At 14 ft. 6 ins. draft.....	4 180	At 17 ft. 0 ins. draft.....	5 539
" 15 " 0 " "	4 450	" 17 " 6 " "	5 815
" 15 " 6 " "	4 272	" 18 " 0 " "	4 180
" 16 " 0 " "	4 990	" 18 " 6 " "	6 385
" 16 " 6 " "	5 264	" 19 " 0 " "	6 680

This ship is considerably larger than the *Victory*. If she were loaded to 20 ft., which is 1 ft. more than her builders have calculated upon, she would carry 7 265 tons. In view of the greater dimensions of this ship, it is doubtful if the *Victory* can carry 7 000 tons. The cost of the ship will be about $250 000. She will have 3 000 H. P. Her operating expenses will be about $175 per day, or $7 29 per hour.

Her fixed yearly expense will be about—

Amortization and interest at 12.5 per cent.....	$31 250
Repairs, painting, etc.....	2 750
Total.....	$34 000

Mr. Dutton. Allowing 15 days for laying up, this is at the rate of $97.14 per day or $4.05 per hour.

At the same rate of speed, etc., as heretofore assumed for the 7 000-ton ship, assuming this ship laden to 20 ft. and carrying 265 tons of fuel, her performance between Chicago and New York will be—

Assuming full return cargoes:

Conducting transportation	17.32	cents per net ton		0.52	cent per bushel.
Amortization, etc	9.62	"	"	0.289	"
Handling cargoes	17.5	"	"	0.525	"
Total	44.45	"	"	1.334	cents "

Assuming one-half return cargoes:

Conducting transportation	23.95	cents per net ton	equal	0.72	cent per bushel.
Amortization, etc.	13.33	"	"	0.4	"
Handling cargoes	17.50	"	"	0.525	"
	54.75	"	"	1.645	"

Returning light:

Conducting transportation	30.725	cents per net ton,	equal	0.92	cent per bushel.
Amortization, etc.	17.06	"	"	0.51	"
Handling cargoes	17.5	"	"	0.525	"
	65.285	"	"	1.955	"

The flat cost of transportation is given separately in order to show that in a life and death competition the big ship is invincible.

The effect of the depth of water upon the cost of carriage is very great in such ships. One inch means, at full draft, 49 tons, or 0.68%, and averages 46 tons, or 0.67 per cent. One foot means 8.4%, and 5 ft. 6 ins. mean 44.7 per cent. At 16 ft. draft the vessel will carry 4 990 tons, or, allowing 265 tons of coal, 4 725 tons of cargo.

The cost would be:

Assuming full return cargoes:

Conducting transportation	25.66	cents per net ton,	equal	0.77	cent per bushel.
Amortization, etc.	14.25	"	"	0.43	"
Handling cargoes	17.5	"	"	0.525	"
	57.41	"	"	1.725	"

Mr. Dutton.

Assuming one-half return cargoes:

Conducting transportation.....	35.49	cents per net ton,	equal	1.06	cents per bushel.
Amortization, etc..	19.75	" "		0.59	"
Handling cargoes..	17.50	" "		0.525	"
	72.74	" "		2.175	"

Returning light:

Conducting transportation.....	45.52	cents per net ton,	equal	1.37	cents per bushel.
Amortization, etc..	25.28	" "		0.76	"
Handling cargoes..	17.50	" "		0.525	"
	88.30	" "		2.655	"

These figures include vessel insurance but not cargo insurance, which runs from 25 to 30 cents per $100 Chicago to Buffalo, and, of course, varies per ton proportionately to the value.

According to the paper, a ton of freight can be carried from Chicago to New York in a 7 000-ton ship for 43 cents; and according to the data here presented, for 44.45 cents. No other means of transportation can approach this for cheapness, and it is evident that if the ship canal be a private enterprise, remunerative tolls can be paid without raising freight costs to the point where competition would begin.

The economy of carriage will take effect, not only on grain, but also on all other commodities, effecting similar savings upon the present rail rates which have ruled this summer for car-load lots from Buffalo to New York, per long ton, as follows:

Upon articles in Class 1.....$7.28.

This class includes many kinds of package freights and small fruits.

Upon articles in Class 2 .....$6.16.

This class includes wool, hops, canned goods, butter, eggs, dressed hogs, green hides, etc.

Upon articles in Class 3.....$5.04.

This class includes cheese, paints, and many kinds of boxed and barreled merchandise.

Upon articles in Class 4 .....$3 36.

This class includes dry hides, baled rope, vegetables, heavy boxed and barreled goods which will stand rough treatment, and certain kinds of castings and metal goods.

Mr. Dutton. Upon articles in Class 5 .....$2.80

This class includes lumber, salt, hay, green fruit, grease, candles, glue, lard, molasses, stove castings, hollow ware, wall finish, peas, potatoes, seeds, etc.

Upon articles in Class 6.....$2.24

This class includes starch, sal soda, rice in boxes, green coffee in bags, beans, etc.

Upon flour, etc.....$1.12.

Upon live stock, as follows, per long ton:

Horses, $6.72; sheep and hogs, $3.92; cattle, $3.70.

The greatest percentage of saving in the cost of transportation will be on dressed meats, which now pay $10.08 per long ton from Chicago to New York, and are carried exclusively by rail. As this business is now conducted, the carcasses are first cooled for 30 days at Chicago and then shipped in refrigerator cars which are re-iced four or five times. When large steamships can ply between the two ports the freshly killed carcasses can be loaded into a ship having a cold storage plant and cooled while in transit, the cost of cooling and transportation being little, if any, greater than the cost of cooling alone at Chicago, because the expenses of the ship would be little, if any, greater per unit of capacity than those of the warehouse at Chicago.

The figures are from the latest official publications of the United States Government. There is a tendency in the articles to slip from one class into another on different roads, which makes exactness of statement impossible without a more searching analysis than the speaker had time to make. It must be borne in mind that over 95% of the freight moved pays these or higher rates, and that to attempt to limit the question to the small movement over the Erie Canal is to lose sight of the true significance to the West of a ship canal to the sea. But the western people do not get their minds side-tracked in any such way. They realize that economy in transportation is their only salvation; that a ship canal to the sea will pay them its entire cost in a single season.

CORRESPONDENCE.

Mr. Lamb.

RICHARD LAMB, Assoc. M. Am. Soc. C. E.—A canal-boat 98 ft. long, 17.5 ft. wide, and of 6 ft. draft, loaded to 240 tons, going at 1 mile per hour on 7 ft. depth of water with a canal prism of 450 sq. ft., meets a resistance of 173 lbs. The same boat, on 8 ft. depth of water, meets a resistance of 145 lbs. On 9 ft. of water the resistance would be about 120 lbs. The resistance increases as the square of the speed, and the power necessary to overcome the resistance as the cube of the speed. It will therefore be seen that on the Erie Canal, with its present depth of 7 ft., a boat loaded to 240 tons, going 4 miles per hour, meets a resistance of 2 768 lbs. and requires 29.44 H. P. to overcome this resistance. If the canal is deepened to 9 ft., the same loaded boat would meet a resistance of 1 920 lbs. and would require 20.36 H. P. to overcome the resistance, or a saving of about 30% in power. The same power that is necessary to propel this boat at 3.34 miles per hour on 7 ft. of water would propel it at 4 miles per hour on 9 ft. of water, a gain of 17% in speed, which at a speed of 4 miles per hour is a saving of about 30 hours on the round trip between Buffalo and Albany over the time at 3.34 miles per hour.

It is not practical to run boats on the Erie Canal much in excess of 4 miles per hour. At a 6-mile speed the water is expelled from below the bottom of the boat. Probably this is owing to the fact that the distance between the bottom of the boat and the bottom of the canal when the boat is not running is about 1 ft., or the boat has about 12 827 galls. of water under her. At 6 miles per hour, 1 166 galls. per second would have to go through 17.5 sq. ft. of sectional area if the boat had not buried any; but as a matter of fact, as the Erie Canal boats are built, practically square in the bow, the boat settles as the speed increases. Hence the sectional area under the boat would be much less than 17.5 sq. ft., making it impossible for such a quantity of water to pass under the boat in such a limited time; hence it takes the easier channels, namely on both sides, until there is no water under the boat at all. This theory is borne out by the fact that at the bows of a canal-boat going at 3 miles an hour an intumescence of the water is formed at least 1 ft. high above the level of the water on the side of the boat. This water, not being able to get under the boat, passes off on either side. In considering the available speed of a canal-boat, the swash has to be taken into account. In a report by John Bogart, M. Am. Soc. C. E., as State Engineer of New York, he tells of a small steam yacht that ran 6 miles per hour on the Erie Canal. They had to stop her from running other than slowly as she damaged the banks excessively. A canal-boat running much over 4 miles per hour, even though the Erie Canal was deepened to 9 ft., would not be able to navigate safely

Mr. Lamb. owing to the many bends and contracted places where steering would be exceedingly difficult and dangerous. As the canal is to-day only 7 ft. deep, a boat drawing 6 ft. cannot be steered at a speed to exceed 4.5 miles per hour, and at this speed only with difficulty and danger. The rudder needs the water to come up from the stern, and when the water practically all goes to the sides, the boat loses her steering way.

It is therefore evident that an average of 4 miles per hour is the proper speed to attain. The steam canal-boats average about 2.5 miles per hour and the horse boats about 1.75 miles per hour. By the writer's system of electric cableway, it is possible to average 4 miles per hour. With the current generated by water power, the cost of this electrical system per hour per mile is far less than either by steam or mules. The writer believes that the Erie Canal boat of the near future will be made of steel, with a spoon-shaped bow to raise rather than depress her when running fast, and will be 150 ft. long, 17.5 ft. to 20 ft. wide and 6 ft. draft. She will be made with a center-board, so that she will not shear when in a sea on the Lakes, and when in the canal the center-board will be raised. These boats will load at the lake ports and be towed in trains by powerful sea tugs to Buffalo; from there they will be towed by electric cableway motors to Albany, and thence by powerful river tugs to New York. All handling and elevator and warehouse charges at Buffalo will thus be saved. The writer believes from his knowledge of the difference in cost of shipping Pocahontas coal by ocean-towed barges over steam or sailing vessels, that if the present cost of shipping grain from Chicago to Buffalo is 1.25 cents per bushel, the train of steel barges can do it for 1.05 cents per bushel. He is also confident that a good profit can be made at barging from Buffalo to New York at 2 cents per bushel by electric haulage. By doing away with the Buffalo handling charges, the rate would become 3.05 cents per bushel from Chicago to New York. No railroad could compete with this rate except at a loss. The author's estimate of probable cost for this service, namely, 2.5 cents per bushel, is most likely accurate and conservative. As for the ship canal *via* Lake Ontario and Lake Champlain, as is proposed by the pneumatic canal-lock syndicate, its estimated cost would make a fixed charge of over 2 cents per bushel on all the freight that would be likely to go through it.

Again, any ship canal that uses the water of Lake Ontario with a view of getting an ocean outlet, would do as well if not better to go *via* the St. Lawrence River, as the steamship route from New York to Europe is northeastward to the Banks of Newfoundland, which track would not be far from the mouth of the St. Lawrence River. If a ship canal was built to New York it would be closed like the St. Lawrence River route with ice during the winter, at which time it would not be earning any money toward its fixed charges.

L. C. SABIN, Assoc. M. Am. Soc. C. E.—When a waterway is made up of two parts, one artificial and narrow, requiring limited speed, the other natural, placing no restrictions on the size or speed of the vessels, the selection of the boat best adapted to the traffic will be influenced by the relative lengths of the two parts and the extent of the restrictions necessitated by the artificial portion. This selection is usually a matter of evolution. It is of interest, however, to endeavor to arrive at the probable cost of transportation over a given route by comparing the conditions in a proposed channel with those existing in channels that are giving certain known results. In making such estimates of cost of transportation between Chicago and New York by the Great Lakes and the Erie Canal, the author reaches what appears to be a rather surprising conclusion. Doubts are frequently expressed as to whether the benefits expected to accrue from the construction of the ship canal from the Great Lakes to the sea would be sufficient to warrant the expenditure of so vast a sum as this project would require, but the proposition is now advanced that a barge canal 9 ft. deep will furnish cheaper transportation than a ship canal 25 ft. deep. Mr. Sabin.

Following the author's figures and assumptions, the cost of transportation between Chicago and New York by tows of four steel barges and one tug, and by the *Victory*, is as follows:

	Victory.	Steel barges.
Running time, round trip	5½ days.	12 days.
Delays in port	13 "	13 "
Total time, round trip	18.5 "	25 "
Round trips per season	11 "	8 "
Expenses "	$120 000	$24 000
" per round trip	11 000	3 000
" per single trip	5 500	1 500
Assumed draft	20 ft.	7.5 ft.
Cargo, tons	7 000	2 400
" bushels wheat	233 000	80 000
Rate on wheat, at per bushel	2.36 cents.	1.87 cents.

If these results represent the true state of affairs harbors may be allowed to silt up if they will; the supposed evolution in size of carriers has in truth been a retrogression, and modern lake freighters must give place to barges of 7.5 ft. draft.

For the consideration of this question there are reliable data concerning the lake traffic, and it would seem well to consult these data for a starting point. The author states that the "transportation between Lakes Superior, Michigan and Erie costs less now than transportation by long ocean voyages; as the vessels have full cargoes both ways of grain, coal, ore, lumber and merchandise." This last state-

Mr. Sabin. ment is not accurate as regards Lake Superior, and probably not so for Lake Michigan. In the past ten years the up-bound freight passing St. Mary's Falls Canal has amounted to but about 30% of the total freight. In other words, the west-bound freight has been less than half as much as the east-bound. The only bulk freight passing into Lake Superior in large quantities is coal. In 1894 the coal passing the canal amounted to only about one-third of the bulk freights, iron ore, grain and lumber, passing east.

It does not appear why "the number and character of the locks, the length of detention in port," etc., "do not affect this discussion." If modern freighters are considered, the assumption of a delay in port of thirteen days for each round trip seems quite unwarranted.

The *Victory* has been so far from spending thirteen days in port for each round trip, that she has made seven round trips from two harbors to Cleveland (or Chicago) in the past two months, or an average of less than nine days per round trip. She therefore went three single trips in 13.5 days instead of lying in port. Large lake freighters do not wait many days for a cargo, but however long they might wait, they could not all carry full cargoes both ways. It is thought that five days' delay in port will be ample allowance for boats receiving and discharging two (one partial) cargoes for each round trip, while 7.5 days might be allowed on the route from Chicago to New York, where partial intermediate cargoes might be taken.

As an approximate estimate of the running expenses of the *Victory*, and the larger size steel barges, the following is submitted :

	Victory. 400 x 48 ft. 16 ft. draft.		Tug and four steel barges, each 180 x 17.5 ft. 7.5 ft. draft.	
Men required per day	20 at $2 00	$40 00	20 at $1 25	$25 00
Subsistence per day	20 at 75	15 00	20 at 50	10 00
Coal, tons per day	30 at 2 50	75 00	10 at 2 50	25 00
Oil, waste, etc., per day		2 50		1 50
Towing charges per day		7 50		
Running expenses		$140 00		$61 50
Running expenses per season	210 at $140 00	$29 400 00	210 at $61 50	$12 900 00
Outfitting, painting, etc.		4 600 00		3 500 00
Insurance	3%	6 000 00	4%	3,200 00
Interest	10%	20 000 00	10%	8 000 00
Deterioration and loss	8%	16 000 00	8%	6 400 00
Total expenses per season		$76 000 00		$34 000 00

In the estimated expenses given in the paper the items are not sufficiently detailed to admit of comparison with the figures given above. In explanation of the large difference in the estimated expenses of run

ning the steel barges, attention may be called to the fact that the author's estimate of $30 per day running expenses would not pay the expenses of the tug alone if running on the Lakes, and the barges must each have at least three seamen aboard. For the sake of argument, it will be assumed that the barges will keep afloat 210 days a year. Mr. Sabin.

Taking the cost of transportation in the Erie Canal, by boats which ply the canal only, at the figures given in the paper, and applying the changes indicated above as to delays in port, probable cargoes and running expenses of lake boats, the following results are obtained:

I.—Ship canal, 18 ft. deep, 16 ft. draft. Steamer *Victory*, 5 000 tons. Total time, round trip, 21 days; 10 trips per season. Expenses per season, $76 000; per trip, $7 600.

Receipts—West, half cargo, 2 500 tons coal at 40 cents	$1 000	
East, full cargo, 5 000 tons wheat at $1 32	6 600	
		$7 600

Rate per bushel on wheat, 4 cents.

II.—Ship canal, 25 ft. deep, 20 ft. draft. Steamer *Victory*, 7 000 tons. Total time, round trip, 18 days; 12 trips per season. Expenses per season (extra coal, $4 000) $80 000; per trip, $6 650.

Receipts—West, 3 500 tons at 40 cents	$1 400	
East, 7 000 tons at 75 cents	5 250	
		$6 650

Rate per bushel on wheat, 2.25 cents.

III.—Present Erie Canal and Lakes.

Chicago to Buffalo, total time, 10.5 days. Round trips per season, 20 days.

	16-ft. Draft.	18-ft. Draft.
Expenses per season	$76 000	$80 000
" round trip	3 800	4 000
Receipts—West, half cargo coal at 40 cents	1 000	1 200
East, full cargo wheat	2 800	2 800
Rate per ton on wheat	56 cents.	47 cents.
Rate per bushel, Chicago to Buffalo	1.68 cents.	1.4 cents.
Elevating and trimming	1.15	1.15
Canal transportation (author)	2.33	2.33
Total cost, Chicago to New York, per bushel	5.16 cents.	4.88 cents.

Mr. Sabin. IV.—Lakes to Buffalo and (90×17.5 ft.) Barges in Canal.

	In Lakes 16-ft. draft.	In Lakes 18-ft. draft.
Chicago to Buffalo, same as III	1.68	1.4
Elevating and trimming	1.15	1.15
Canal transportation (author)	2.00	2.00
Total cost per bushel, Chicago to New York	4.83 cents	4.55 cents.

V.—Steel barges, 180×17.5 ft., Chicago to New York.

	Five days delay; half cargo coal west on Lakes.	7.5 days delay; half cargo mdse. on canal at 50 cents, half cargo coal on Lakes at 40 cents.
Total time, round trip	28.5	31.
Round trips per season	8	7
Expenses per round trip	$4 250	$4 860
Receipts—West	480	1 080
East	3 770	3 780
Rate per bushel on wheat, Chicago to New York	4.7 cents	4.7 cents.

Summarizing these results the following table is obtained:

Case.	Conditions.	Dollars per ton.	Cents per bushel.	Mills per ton-mile.
I..	Ship canal 18 ft. deep, allowing 16 ft. draft	1.32	4.00	1.00
II..	" " 25 " " " 20 " "	.75	2.25	0.55
III..	Barge canal 7 ft., and Lakes { Lakes, 16 ft. draft	1.72	5.16	1.25
	" 18 " " }	1 63	4.88	1.20
IV..	Barge canal 9 ft., and Lakes { Lakes, 16 ft. draft	1.61	4.83	1.18
	" 18 " " }	1.52	4.55	1.12
V..	Barge canal and Lakes, no transshipment	1.57	4.70	1.15

Comparing I and II, the advantage of the deeper canal is clearly shown, but if I and II are compared with V, it will be found that even an 18-ft channel is, according to the assumptions made by the writer, preferable to a barge canal, while the 25-ft. channel would effect a saving of nearly 2.5 cents per bushel over transportation by barges.

In conclusion it is of interest in passing to note the amount of tonnage required to pay interest on an investment of $150 000 000 in building a ship canal 25 ft. deep. Reckoning interest at 4%, the amount will be $6 000 000 per year and with a saving per ton of ($1.57—$0.75) 82 cents, the required freight tonnage would be about 7 500 000, or about one-half the amount that passes St. Mary's Falls Canal.

L. J. Le Conte, M. Am. Soc. C. E.—The author's premises will hardly stand fair criticism, and, as a natural consequence, his final conclusions must be more or less subject to some modification. He seems to be in error in assuming that the estimated expenses in the two cases relating to ship canal will be $116 450 and $120 000 respectively. These estimates are plainly based on the supposition that the entire lake fleet of large vessels will always remain hibernated in the Lakes for five months in the year, as is now the case. On the contrary, so soon as the ship canal shall have been completed, this lake fleet will find it to its interest to migrate every winter to the Atlantic and engage in the coasting trade, where it will remain until the season on the Lakes opens again. It is clear that the second and third items under estimated expenses will be reduced practically to one-half the amounts stated in the paper. These cases will thus stand as follows: Mr. Le Conte.

I.—Eighteen-foot Erie ship canal. Total annual expenses on lake and canal traffic, $84 500; rate expressing actual cost of transportation, 3.67 cents per bushel.

II.—Twenty-five-foot Erie ship canal. Total annual expenses chargeable to lake and canal traffic, $87 000; rate expressing actual cost, 2.36 cents per bushel, which is something less than the author's barge-train rate, 2.5 cents per bushel.

The author's remarks relating to the introduction of a system of barge trains, 2 400 tons each, with a 9-ft. canal, are certainly wise and economical from every point of view, but it is evident that the ship canal will accommodate both ships and barge trains alike. Furthermore, the 25-ft. ship canal will admit of deep-draft barges and trains of much greater length, controlled by two tugboats, one at either end, so as to have complete command while going around the bends. As a result, greater speed and economy will certainly follow. The actual cost of transportation of grain by these large barge trains ought not to exceed 2.25 cents per bushel from Chicago to New York City.

Lewis M. Haupt, M. Am. Soc. C. E.—The question of cheaper transportation is always pertinent, and it is generally admitted that the open waterway furnishes the cheapest known medium. When it comes to restricted artificial channels, however, it is evident that a limit may be reached beyond which this law does not obtain. The author has shown in a practical and conclusive manner, as based upon his data, the relative importance of the barge canal and the apparent inutility of a ship canal between the Great Lakes and tidewater. But this case can hardly be used as a basis for a generalization since his figures are concrete. They serve to show, by inference at least, how many elements there are entering into the problem, and especially the importance of time, as measured by distance, lockage, demurrage and sectional area. He also shows an intimate relation between vessel and Mr. Haupt.

Mr. Haupt. channel, and raises the question as to whether the canal should be designed for the ship or the reverse. Here, again, arises a question of policy as to what size of waterway will yield the largest returns for the capital invested, which is a function of traffic, rates, geology, topography, labor, prices, plant, water supply and many other variables. But for a given traffic, at a paying rate, it will be a simple matter to determine the capital which may be safely invested, while the size of the trunk is more largely dependent upon water supply than on any other factor. This determined, the dimensions of locks and consequently of boats is in a measure fixed by the ratio between midship section and cross-section of the waterway necessary for appropriate speed without material damage or excessive power. The mere clearance under the keel is not so important as the width, for, in the writer's opinion, it is the inequality of pressure due to the lateral wave displacement in narrow channels, when the boat is not in the center, that causes shear.

The allowance made for port detentions in the calculations seems excessive, especially in view of the mechanical appliances for handling cargoes in bulk. The statement has been made that a lake propeller unloaded 3 000 tons at Erie, sailed to Ashtabula, and took on 1 800 tons, entering and clearing at both places in one day.

The present demands of lake commerce may be economically met by the 9-ft. barge navigation, but it should be remembered that the 14-ft. outlet through Canada will soon be completed, and if the commercial value of a waterway varies, according to Stephenson, as the cube of the depth, where will New York and the Atlantic seaboard come in, at this ratio of 9^3 to 14^3, or about 1 to 4?

Improvements in canal lifts and locks are now in order, and the prospects are that ere long the old gate of Leonardo di Vinci will be supplanted by the hydraulic lifts, which save both time and water, thus effecting still greater economy in the much needed improvements in internal navigation.

The author's statements hardly apply to the proposed coastwise canal system, nor are they intended to do so, since the vessels need not be constructed for a particular class of trade. These canals should be capable of passing any ordinary coaster, whether barge, brig, steamer or canal boat, at a fair rate of speed, and it is found that they will save at least 50% of the time on the round trip, and can be operated at a toll of less than half the present rates, and with benefits to all interests. Yet it is asserted that they are not needed, would not pay and will never be built.

Mr. Babcock. W. J. Babcock, Esq.*—The use of steel barges 180 ft. long, 17 ft. wide and with 7.5 ft. draft in a safe, large and uninterrupted traffic between Chicago and New York, is utterly impracticable from any commercial standpoint. The writer could guarantee to build a barge of these dimen-

*Manager of the Chicago Shipbuilding Company.

sions which would be entirely seaworthy and safe for the lake trip in all weathers, provided the towing vessel was either a large steamer or a powerful tug, and not more than two or three such barges were in one tow. If the power is to be applied in a similar barge, however, intended to go through the canal herself, then the experiment would be dangerous, as the draft of the tug is too little, and she could not make headway against a gale or sea with a tow behind her, and hardly by herself even. Such a barge, without power, would carry 17 000 bushels of wheat. There are boats on the lakes which carry ten times that amount even under the present conditions of navigation, and make their trips with almost the regularity of a passenger train. Ten barges would require four or five steamers to tow them, and the whole combination could not begin to transport the 170 000 bushels as cheaply as the one boat would do it. Mr. Babcock.

THOMAS T. JOHNSTON, Esq.*—The paper states that "the loud cry for ship canals" is because of the charge for transshipment of cargoes at Buffalo, but there are broader grounds for it. The ship canals at the mouth of Lake Superior are the most important in existence. It is within the memory of men still young when the canal and locks at St. Mary's Falls were of 9 ft. lift, two in number, with a depth of 12 ft. on the mitre-sills, an arrangement permitting a much more extensive commerce than would be possible in the Erie Canal enlarged to a depth of 9 ft. The usefulness of such an arrangement ceases when the tonnage exceeds about 5 000 000 tons. To-day there are two canals and three gigantic locks, those of recent construction having a depth of 21 ft. on the mitre-sills. The barrier to commerce at this place could not possibly be surmounted by a canal having even 14 ft. depth, and hence the loud cry for ship canals at this place. Mr. Johnston.

Before a ship canal could be constructed from the Great Lakes to the sea, great changes will take place in the commercial requirements of the people affected by the Great Lakes as a commercial artery. St. Louis, through the Chicago sanitary and ship canal, will stand in the same relation to the Great Lakes that Duluth does now, and 50 000 000 people west of Chicago will use foreign goods brought to them by way of the Lakes. The interchange of commodities east and west will be many times greater than at present, and individual products will be demanded in such large quantities that bulk shipments can be made. Small craft and canals for small craft cannot carry the traffic, and hence another loud cry for ship canals.

"The question of first cost, the interest on which is represented by tolls," is at the bottom of much opposition to ship canals. If the sentiment on which this matter of tolls rests is to prevail, the canals and locks at St. Mary's Falls should be destroyed at once, and no man should be allowed to drive his wagon from farm to market without

* Assistant Chief Engineer, Sanitary District of Chicago.

Mr. Johnston. making a cash payment for each trip. Existing transportation lines, aside from waterways, cannot possibly carry the commerce of the future, and must prosper in general as a consequence of the development of ship canals. The facilities for commerce offered by a water route, free from tolls, affords the basis for another loud cry for ship canals.

It is assumed in the paper that the *Victory* will be delayed in port $\frac{13}{29}$ of the time she is in commission; that a ship canal from Buffalo to New York will have 50 locks; that 300 to 600-ton barges can be made suitable craft for lake navigation at an expense of $6 a day; and that it is proper to settle the merits of a ship canal on the basis of grain transportation alone. It is a matter of record that a large lake vessel has delivered a cargo of ore, been loaded with coal, and cleared within 15 hours. The ship canal from Buffalo may have less than a fourth of fifty locks. The harbor of Chicago could scarcely hold enough 600-ton boats to deliver and load the grain and flour shipments that are made now, to say nothing of the future. There are many cargoes to be carried between the Great Lakes and the seaboard which cannot be correlated to grain cargoes.

The assumptions on which the calculations in the paper are based may be easily paralleled with other equally logical assumptions leading to very different conclusions. The merits of a ship canal from Buffalo to New York may be logically discussed in the light of the performance of the best class of boats carrying through freight. At the present time every existing transportation facility between the Upper Lakes and the seaboard is, and for some time will be, taxed to its utmost capacity to handle commerce. The city of Chicago will this fall pay a tax of more than $2 000 000 on anthracite coal alone on account of this state of affairs.

Mr. Clarke. T. C. Clarke, M. Am. Soc. C. E.—The object desired by the author "of exciting discussion," has certainly been attained. The conclusions of his paper were:

First.—"That a ship canal ought not to be less than 25 ft. deep, if it be intended to pass large lake vessels." None of those discussing the paper disagree with this.

Second.—"That a slight enlargement of the Erie Canal may allow fleets of steel barges to run from New York to Chicago, and transport freight at less rates than large lake vessels could do in a 25-ft. ship canal, even if it were (also) used free of tolls."

This statement is controverted, but the author fails to see that his arguments have been disapproved. The points attempted to be made are:

That the author has allowed too long a time for detention in port, and to disapprove this several writers show that the large lake steamers carrying coal to and ore from the upper lake ports to Chicago or

Cleveland only remain one day loading and unloading at each port. Mr. Clarke.
This fact was well known to the author, but it has nothing to do with the case of a ship canal through the State of New York. A large lake steamer gets her extraordinary economy of transport because she is moving back and forward all the time with full cargoes between two terminal ports. As she can get full cargoes she does not have to stop by the way.

Put this expensive machine on a canal, and all the conditions are changed. Her rate of speed cannot be over one-half and possibly not over one-third of that in deep water. New York and any single lake port could not afford full cargoes. She must stop at intermediate ports, both on the canal and on the Lakes, to take on and discharge cargo, or she will not have enough. Hence the author allowed 13 days for detentions in ports and still believes it a correct and conservative estimate.

As to the details of cost of operation while in motion, all those discussing the paper differ from each other as well as from the author, but he sees no reason to change his figures.

The ship canal would enable the lake fleet to go to the sea, and carry on transportation between Atlantic ports during the season of closed navigation on the Lakes, during such time as they would not have to be laid up for repairs. But the same advantage can be gained by fleets of steel barges, which could be used between Boston, New York and Philadelphia, as similar craft are now used. The author fails to see how his comparison is affected.

Mr. Babcock says that a barge 180 ft. long, 17 ft. wide and $7\frac{1}{2}$ ft. deep is impracticable commercially, because a large steamer or powerful tug would be required on the Lakes, more powerful than that necessary on the canals. The author fails to see why one kind of tug could not be used on the Lakes, another on the canals.

But the author did not say that such large barges were absolutely necessary; he merely suggested (page 5, line 2) that it might be easier to get insurance upon them than upon smaller barges. This matter of insurance really is the key to the whole problem, and none of the objectors have denied its possibility. The company which built and tested steel barges on the canal and Lake to Cleveland do not seem to fear any trouble, as they have lately given an order for ten more barges and three tugs.

From the reports of the Inland Lloyds for 1895 it is possible to analyze the cause of disasters to the lake fleet. It is generally believed that storms and rough seas are the cause of the greater part, and that fleets of barges are not as seaworthy as larger steamers. This is true, but it is interesting to see how small a part of the actual losses are due to winds and waves.

Mr. Clarke. GRAND TOTAL OF LOSSES ON BOATS ON UPPER LAKES. FROM REPORTS OF INLAND LLOYDS IN 1895.

1.	Collisions	32%	$667 369
2.	Driven ashore	22%	476 914
3.	Disasters in the Sault Ste. Marie passage	14%	301 429
4.	Fire	16%	346 445
5.	Struck rocks	12%	211 985
6.	Unclassified	4%	70 777
		100%	$2 077 697

A very cursory inspection of this table shows that on items 3, 4 and 5 the chances of disaster would be less to tows of barges than to large steamers, while on item 1 it need be no greater. On item 2 only the barges would be more liable to disaster than the large single craft.

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No. 771.

A HIGH-SPEED GRAVITY FILTER BED.

By A. McL. HAWKS, Jun. Am. Soc. C. E.

PRESENTED SEPTEMBER 4TH, 1895.

WITH DISCUSSION.

The following description of a filter bed designed and constructed for the Tacoma Light and Water Company by the author, as engineer in charge, may prove interesting because of the rapidity of construction, extremely low first cost, ease and cheapness of operation, and the excessively high speed at which it was operated. On May 12th, 1892, the author was consulted by the chief engineer of the Tacoma Light and Water Company concerning the necessity of purifying that portion of the water supplied to the city of Tacoma which comes from brooks situated about 10 miles south of the city upon the gravel prairie, and the means to be used to obtain such purification.

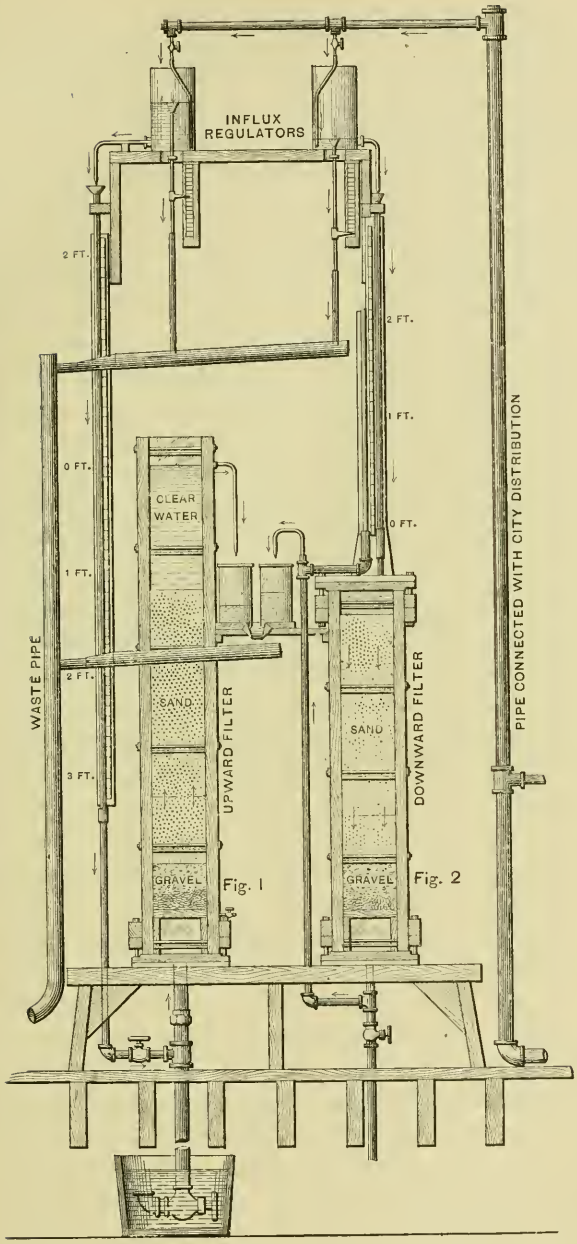
In order to understand the demand for such a move on the company's part, it will be necessary to explain the causes and surroundings. Bushalier Creek, from which the company took its upper supply, is the outflow from a small lake called Spanaway, which has a surface area of 260 acres, and is supplied from springs in and about the lake

and from one main feeder 5 miles long. The normal flow of Bushalier Creek is from 5 000 000 to 7 000 000 galls. in the dry season, rising to 20 000 000 or 25 000 000 galls. in the rainy season. On the banks of the lake are several farms and a town, Lake Park, with 500 inhabitants, and the whole length of the feeder is through farms and tilled land. Although the houses stand upon a gravel plain 10 to 50 ft. above the lake level, which would act as a rough filter of that depth, contamination steadily increased until the spring water from the gravel beds had become very offensive in the dry season; added to this was the growth of algæ and other low forms of animal and vegetable life which thrive wonderfully on the Pacific Coast, and when dead and decaying give a particularly rank odor and taste to the water.

At the time the author was called upon, these causes had not operated sufficiently long to demand a very elaborate or costly plant; and, too, the company had already purchased Maplewood Springs, having a minimum flow of 11 000 000 galls., and proposed to purchase others, so that ultimately the whole supply would come from pure sources beyond possible contamination. The whole matter was therefore regarded as a temporary expedient to last but a few years and to be used only during the summer months, and economy of construction and operation, as well as that bane of all engineers' lives, great haste for completion, were a desideratum.

Mechanical filters were considered, but the time required to negotiate for such a plant, as well as the first cost, debarred them; the Anderson process was seriously considered, but cost also barred this. To grade and prepare the requisite space in which to operate a gravity filter at 50 galls. per square foot upon the system in use at Lawrence, Mass., would cost $2 200, and for roofing over such an area, an absolute necessity in this climate, the cost would be $3 000. These two items were sufficient to put this out of the question. There remained practically but one thing to do, and that was to design a new form of bed which could be operated at a much higher speed than usual.

While a student in the office of J. Herbert Shedd, M. Am. Soc. C. E., the author studied the "Report on the Filtration of River Waters," by the late J. P. Kirkwood, Past President Am. Soc. C. E., and in leisure hours made many experiments, using as his original plant some very large flower-pots, graduating later on to two lengths of 8-in. drain tile. At a still later date, while employed in the office of the water-


INFLUX REGULATORS
2 FT.
0 FT.
1 FT.
2 FT.
3 FT.
WASTE PIPE
CLEAR WATER
SAND
GRAVEL
UPWARD FILTER
Fig. 1
2 FT.
1 FT.
0 FT.
SAND
GRAVEL
DOWNWARD FILTER
Fig. 2
PIPE CONNECTED WITH CITY DISTRIBUTION

works commissioner of the city of St. Louis, the experimental filters then being run furnished much valuable information. The two small filters, Figs. 1 and 2, had sides made of plate glass, thus showing plainly every action which took place within them. They were run at the regular rate of 50 galls. per square foot per day.

It was noticed in their operation that a head of 0.1 ft. was sufficient to run them for some time; as they clogged, this head increased to 3 ft. for the downward type, and 1 ft. for the upward. When this point was reached, they were cleaned by removing the top layer of sand in the former and reversing the flow in the latter, after which they would be started again with the same head as at first. If much greater head was put upon them a break would occur, forcing the sand down into the gravel in the downward type, or, passing up in the other, would agitate the sand in the manner which is seen in every boiling spring, in both cases destroying the integrity of the filter and distributing the impurities all through the sand. Other minor puzzling questions were solved by the glass filters, but this was the most important, as it explained why no more than about 100 galls. per square foot could be forced through the tiles without seriously affecting their filtering qualities. These glass filters suggested one important question: If a head of 3 ft. could be safely imposed on them without injury, simply because a thin coating of impurities acted as a damper to retard the flow and reduce the pressure, thus preventing disturbance of the sand, why could not some means be devised to hold the sand in place and make use of a part of that head for operation, running them at a much higher speed? Accordingly it was decided to attempt in these beds to run at a speed of 500 galls. per square foot per day. To do this work required a sand of large grain, free from all finer matter, and there was the danger that the chief usefulness of a filter, viz., its ability to thoroughly strain out all impurities might be lost. To avoid this and to help hasten the process, it was determined to use a coagulant. Three of the cheapest and readiest were considered: alum, borax and iron in the form of oxide or carbide. The first two are comparatively expensive and are open to the further objection that they add a certain hardness to the water and are considered unhealthy by many. The third is very cheap and lasting; and if a small quantity enters the filtered water it only acts as the rusty nails which it was formerly thought necessary to keep in the water-pail to have the very best water.

It was decided also to have some form of aeration in connection with the filters. Much has been written on both sides of the case concerning the value of entrained air as a factor in the purification of water. In intermittent filtration, where the periods of submergence are short and at infrequent intervals, it is doubtless one of the greatest factors; but in filters which are in constant use, where the exposure to the air is only for an hour or two every few days, or possibly weeks, not much can be expected from this cause. As a matter of fact, the glass filters at St. Louis showed no sign of holding entrained air at any time; and as will be shown later on, the Tacoma filters showed no sign of its presence 15 minutes after the water was turned on. That oxygen is one of the best purifying agents known, no one will deny; that such a small amount as could be retained and not be found in a filter so exposed as this was could be of value may be doubted. In order that air may act beneficially either the water must be thoroughly exposed to it in the form of thin sheets falling over weirs, or else the air must be introduced and thoroughly mixed with the water in the form of minute bubbles. As this latter method required the installation of some form of air compressor, it was passed over and the first was used.

With the foregoing premises in mind it was determined to construct a filter having these main features, viz., screens at the headgate to remove all large trash; a flume of sufficient dimensions to give at least 10 minutes' time to operate on the water while it was in transit; grade to make five overfalls, and as many slopes to agitate the water; five level sections to hold scrap iron and punchings; and, finally, an upward filter in which to remove all impurities; to be roofed over, and to operate at the rate of 500 galls. per square foot per day.

On May 14th the author made a rough survey of the locality where the filter was to be built and immediately commenced the plans. "Rapidity of erection and economy of construction and operation" were the instructions most constantly urged, and too often were made the limit to which all arguments were brought, to the detriment of the operation of the filters. Good work was done by them; better could have been, had this motto not been brought forward so frequently and had a little more money been expended in filling out the plant and making it in strict accordance with the best ideas.

An old saw-mill acquired by the company when they purchased the water right was used as a shed in which to install the beds. The old

flume leading to town passed a few feet to one side of this, the creek about the same distance on the other. The old diverting dam at the head of the flume was 430 ft. distant from the mill and at such an elevation that by adding about 3 ft. to its height and excavating to the mud-sills of the old mill, the flume could be built to the desired grade, with the overfalls as planned. All these things combined to make a very economical construction. The cut to be made for the flume was entirely through gravel, which furnished most of that material required in the filter save the smallest size. Sand of a suitable quality was not to be had at a nearer point than a pit 3 miles distant, which made the price laid down at the mill $1.25. When this sand was thoroughly washed and screened it made but $\frac{2}{3}$ cu. yd., so that the price could really be called $1.80, inclusive of labor in washing.

On May 17th plans were completed and the bill of material given to a neighboring mill to be filled. The next day men were put on the ground and the actual work of construction commenced. The total bill for the tools, exclusive of those used by the carpenters, was about $100, of which about $50 was credited to the work upon completion. The cost of different portions of the work was segregated for use in future work of similar character. Laborers were paid $2 per day and carpenters $2.75 to $3.50; teams cost $4 where used on the work, though most of the teaming was on small contracts.

The diverting dam required 6 000 ft., B. M., of timber, worth $72 in place, and 200 cu. yds. of earth and gravel at 20 cents per yard; part of the face was paved at a cost of $15.

The headgate called for 500 ft. of lumber worth $15 in place, and it cost $10 to excavate and backfill the site. The screens cost $5. The flume cost 40 cents per linear foot for lumber and erection, or $172; the excavation and filling along the whole 430 ft. cost $275. The scrap iron weighed 8 500 lbs. and cost $65. The cost of clearing out the gravel and refuse in the old mill was $75, and the repairs and alterations $125. The cost of lumber in place to construct the beds and house flume was $180. The gravel for the filters cost 35 cents per cubic yard washed and in place in the beds. After 45 cu. yds. of sand had been hauled from the pit to the filters, a deposit was found near the headgate. Although all material taken from this latter pit had to be replaced by gravel from the cut, because it was so near the headgate and the edge of the pond, it cost less to excavate, screen, wheel into the head

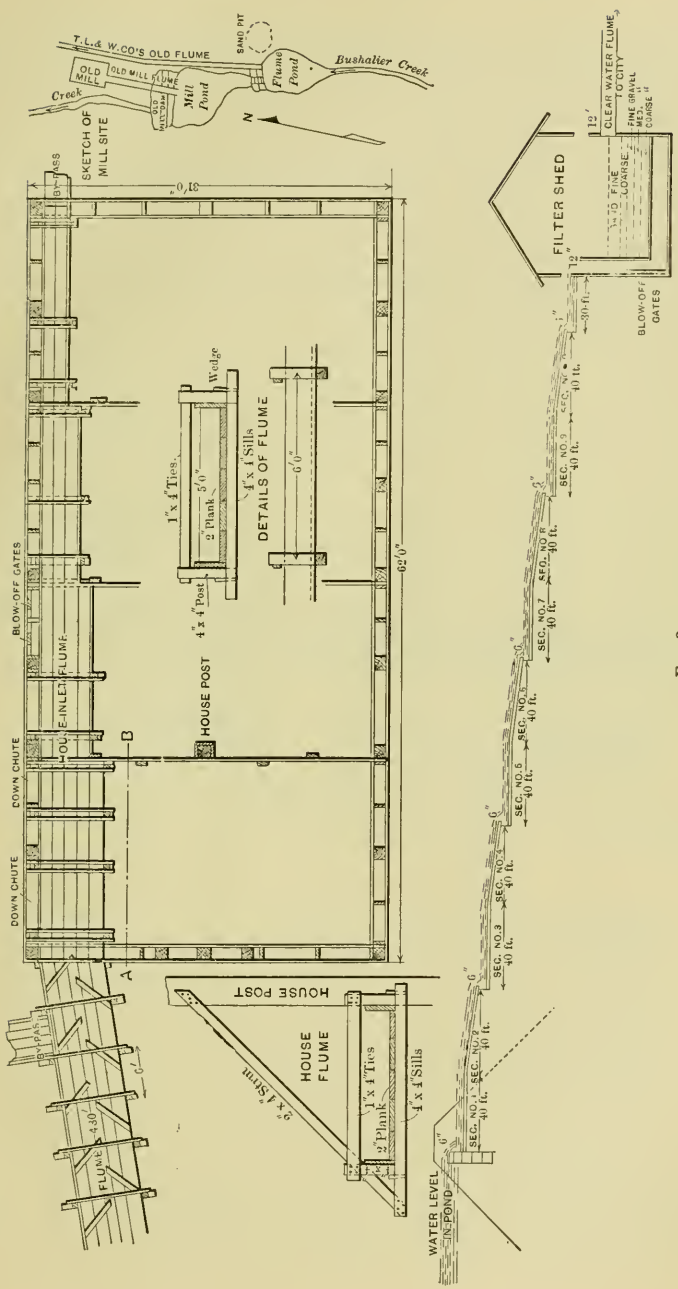

T. L. & W. CO'S OLD FLUME
SAND PIT
Bushalier Creek
Flume Pond
Mill Pond
OLD MILL
OLD MILL FLUME
Creek
N
SKETCH OF MILL SITE
BY-PASS
31'0"
62'0"
BLOW-OFF GATES
DOWN CHUTE
DOWN CHUTE
INLET FLUME
HOUSE POST
4" x 4" Post
Wedge
1" x 4" Ties
2" Plank
5'0"
1" x 4" Sills
DETAILS OF FLUME
6'0"
HOUSE POST
HOUSE FLUME
9" x 4" Strut
1" x 4" Ties
2" Plank
1" x 4" Sills
FLUME
4'0"
WATER LEVEL IN POND
FILTER SHED
CLEAR WATER FLUME TO CITY
FINE GRAVEL
COARSE
BLOW-OFF GATES
SEC. NO. 1
SEC. NO. 2
SEC. NO. 3
SEC. NO. 4
SEC. NO. 5
SEC. NO. 6
SEC. NO. 7
SEC. NO. 8
SEC. NO. 9
40 ft.

Fig. 3.

of flume, wash down to the filter and put in place than to prepare the first lot. One reason for this was that the water in the flume both washed and transported it to the house; two men on this part of the work handled over 20 cu. yds. of cleaned sand a day after some "sand hustlers" were made for them. These were simply boards with handles nailed to them, so the men could walk alongside the flume and direct the current of the water as they pleased by partially damming the stream with the "hustlers." The dirt and finest sand would travel most quickly and be wasted out into the creek at a by-pass above the house; then the main part of the sand would follow, flowing into the house as the water was drawn off through the by-pass at the lower end of the house. In this way thoroughly washed sand was delivered to such a point that it could be cast into place at one handling.

Beside the ordinary repairs and alterations it was decided to make a considerable part of the sides of the mill in the form of a blind lattice in order that air might have free access and yet sunlight and falling leaves be excluded and danger from outside interference be lessened. This cost $65.

Repairs costing $50, necessitated by the construction of this filter, were made to the old flume and headgate.

The total cost of the filter bed was as follows:

Tools and implements	$50 00
Clearing at dam	25 00
Constructing dam	127 00
" headgate	30 00
" flume	447 00
Repairs and alteration of old mill	200 00
Carpentry of beds and house flume	180 00
100 cu. yds. gravel	35 00
30 cu. yds. sand	54 00
140 cu. yds. sand	140 00
Latticing mill and old flume repairs	115 00
Total	$1 403 00

This may be safely taken as "economy of construction."

The work was completed on June 29th, or in 37 working days from the day men were first put on the ground, which certainly filled the requirement of "rapidity of execution." The construction of the plant is shown in Figs. 3 and 4. Water was turned through the beds at once and allowed to run until no sign of turbidity was visible. The headgate was then closed while the iron was being stacked in the level sections of the flume and the diagonal strips in the slopes. Before the iron was placed it was washed and roasted, to remove any dirt or machine oil which the turnings and punchings had retained. The stacking was in the shape of windrows across the flume every 5 ft., leaving a space at the head and end of every level section. Water was then turned on, the filters filled and blown off once more

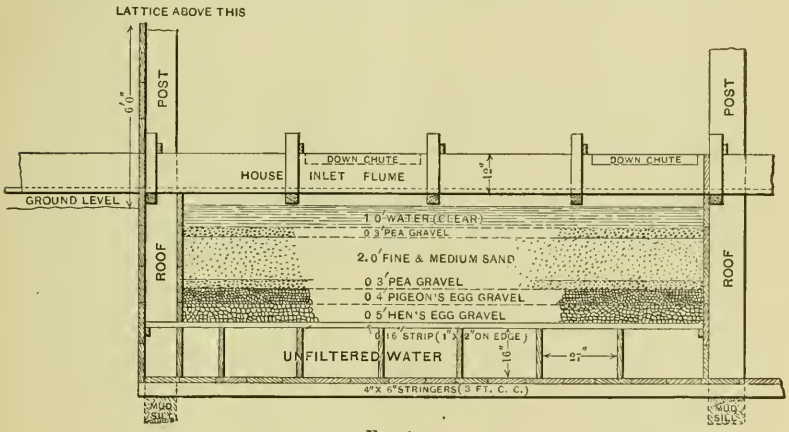


FIG. 4.

and then put into actual operation. This process of cleaning an upward filter by blowing off was very simple and was done easily. The gate at the head of the house flume was closed, that of the by-pass adjacent being opened at the same time, and the water wasted into the creek; the blow-off gates at the side were opened wide, allowing the water to flow quickly out from beneath the sand, and that from above to reverse its direction of flow, and, pushing back, to wash the impurities from the lower face of the sand. The blow-off was then closed, the house gate opened, the by-pass closed, and water once more admitted to the filter. The time required to empty the filter was 5 minutes, so the speed of the descending current was more than 10 times its usual up-

ward flow, which should be current sufficient to remove any particles held against the face of the sand. Water was admitted as rapidly as the flume would permit until the "tell-tale" showed it had risen to the pea gravel beneath the sand; then it was reduced to its normal flow, and in 40 minutes after the gates were opened the filter was in full operation.

For several weeks much of the author's time was devoted to operating these beds, and great care was taken to maintain them at their best. While they were being constructed causes had arisen to compel the speed of operation to be raised to over 700 galls., and times were frequent when 1 000 galls. were passed successfully without disturbing the materials or destroying the integrity of the plant.

The first change to be made was in the removal of the turnings of steel from the iron, on account of the tendency of the rows to rust into a solid mass and so cause the water to run over the top instead of passing through the rows. Next came the removal of the punchings from a like cause, and finally all scrap was thrown out and old horse shoes substituted throughout. These last made the best rows, were more easily handled and more satisfactory in every way. The matter of handling became of some moment, as it was necessary to change the rows every second or third day after operations had been carried on for a few weeks. The iron seemed to be a great attraction to all sorts of living creatures in the water, and in two days after being placed in the flume a shoe would be covered with insects to such an extent as to impair its usefulness seriously. New rows were then placed beside the old ones, the latter removed to a drying heap, where they lay in the open for a day, and then a big fire of brush made about them and they were roasted. More than a bushel of remains was gathered at the drying place in one month. This was one of the unexpected features of the work and one the author has never seen mentioned elsewhere. It was supposed when work commenced that renewal of the iron every month would be sufficient, in fact no extra supply had been prepared to replace the first lot, which was an economy, as later events proved; for in all likelihood, such an extra quantity would have been in the shape of turnings, punchings and scrap, which would have been useless.

The next trouble came from the screens at the headgate, which were made from such copper wire screen stuff as could be purchased

in town. For a little time this did fairly well, but as soon as the temperature of the water in the lake began to rise, the algæ began to grow and float down stream, and becoming entangled in the screens, were very difficult to remove; in efforts made to remove them the screens would become badly misshapen and require renewal. As the author had used perforated copper and brass plates in Colorado under similar circumstances he purchased some from the Harrington and King Perforating Company of Chicago, at a cost of $44, which served the purpose perfectly and soon saved the seemingly large first cost in the time and labor formerly expended in cleaning, beside all cost of renewal, as these were never worn out or broken. Probably nine-tenths of the time and energy formerly expended upon the screens was saved, and as they were cleaned five to seven times a day this meant considerable to the man in charge.

In blowing off these filters, of course most of the water drops out of the sand and its place is at least partly occupied with air, which is drawn into the partial vacuum in the sand bed. This is the case much more than with any downward filter, especially when this latter in time of cleaning has the water drawn down only a little below the surface of the sand, to remove the top layer. When the beds were refilled a good opportunity was given to watch the effect of the entrained air; as the water reached the surface the greater part seemed to rise with it; for 5 minutes quite an ebullition went on; gradually this ceased until at 10 minutes it had practically stopped; and at the end of 30 minutes no sign could be found in the beds of its having been present. This seemed to refute the idea that entrained air was of any practical value in this type of filter.

The iron acted rather slowly as a coagulant, and the water was not as thoroughly aerated as was desirable; but the combination of these two was thorough if only sufficient time was given. Samples of the water taken as it was discharged from the house flume showed the presence of the iron in a faint milky tinge; samples which were well shaken rapidly increased in this, and some bottles which have been kept more than two years show the bottom third as if filled with liquid opals, but the water has no smell or taste, nor has it at any time undergone anything like fermentation.

After running this plant for some weeks, during all which time it was operated at a higher speed than designed for, the company decided

to erect the second plant upon similar plans at Clover Creek, and a care-taker was put in charge. So the management fell into other hands and for some months the author had neither time nor opportunity to observe its further workings. After that the water and electric light plants were sold to the city; and at the same time a farmer lower down on the creek took out an injunction to prevent the operation of blowing off the filters, as it destroyed the purity of the stream where it flowed over his land. So the plant has not been operated for some months, and there seems to be no likelihood of its ever being used again to continue these investigations.

From what was seen of the operation of this plant, the author was led to believe that a most successful plant could be built by observing the following rules, viz., to have the screens at the headgate so arranged as to remove all large matter; to thoroughly aerate the water before it reaches the iron as well as afterwards, as in that case the oxide is more freely given off by the iron and absorbed by the water; to so place the iron that the water is compelled to come into thorough contact with it, and that it be agitated in the water, as well as be capable of ready removal and replacement; to give sufficient time for the air and iron to act upon the water before subjecting it to filtration; and finally, in the filter itself, the sand should be most carefully graded and the changes from pea gravel to the finest sand be made carefully and evenly. Every detail of construction and all materials should be rigidly inspected, and every item should be watched carefully in operation.

With these points thoroughly worked out, a filter operating at the rate of 500 galls. should be a commercial success as well as a mechanical one. The first cost of the beds depends greatly upon the structure of the house in which they are installed; for the beds alone $1 50 per square foot should be ample; for roofing the same, 35 to 50 cents ; for flume with weirs, iron, etc., $1 per linear foot should be estimated; if aeration by jet of air be used the cost of such a plant must be added. In operating, the services of one man on each shift should be sufficient for a 5 000 000-gall. plant, provided the iron can be changed readily; for such work the man must have some idea of mechanics, and would be well worth $75 per month. The cost of raising the water, say 5 ft., for overfalls, or of running a small air compresser, would not be more than $2 per day.

The total cost of constructing a 5 000 000-gall. plant would be about as follows, allowing 10% excess for cleaning purposes :

Excavating site, 11 000 sq. ft., 7 ft. deep, about 3 000 cu. yds., at 20 cents	$600 00
Concrete floor of same, 400 cu. yds, concrete at $5.	2 050 00
Rubble masonry walls—	
420 ft. outside walls, at $10	4 200 00
560-ft. division walls, at $8	4 480 00
Flume for intake, 800 lin. ft., at $1	800 00
Filter beds, 11 000 sq. ft., at $1 50	16 500 00
Roofing for same, at 50 cents	5 500 00
Blow-off gates, pipes, etc.	2 500 00
Total	$36 630 00
Add for contingencies	3 663 00
" " engineering and superintendence	2 500 00
Total	$42 793 00

The cost of operation would be—

Two men, at $75 per month	150 00
Cost of pumping, extra 5 ft. elevation	60 00
	$210 00

This makes the cost per 1 000 000 galls. (150 000 000 galls. per month), $1.40. To this must be added interest on the investment, and a small amount for renewals, everything being contemplated to be of the most durable character.

DISCUSSION.

Mr. Hering. RUDOLPH HERING, M. Am. Soc. C. E.—About six months after the filter described in the paper had been put in operation, the speaker make an examination of it. The applied water was objectionable mainly on account of the very large number of infusoria it contained and not on account of any sewage pollution. The wash water from the filter, after the current was reversed for cleaning the beds, contained myriads of various varieties of small organisms. No such life was detected in the filtered water. It was practically all removed during the passage of the water through the fine sand. The filter acted principally as a strainer to remove such organisms. Apparently the iron did not have much effect on the results, although it produced a little coagulant. Owing to the fact that no chemical or bacterial analyses had been made to determine what the filter actually accomplished, the speaker considered it impossible to judge of its value other than as a strainer, in which role it was successful.

In the correspondence (see page 59) on the paper, Mr. Hill implied that typhoid fever deaths are due wholly to water supplies, to which statement the speaker took exception. At Atlanta, Ga., to which city Mr. Hill makes a direct reference, the speaker believed that the typhoid fever death rate cannot be due wholly, if at all, to the water supply. The former supply was drawn from a small area on which there was a very small population, and the entire territory was kept under very strict supervision by the local board of health. The new supply is drawn from a large stream, the Chattahoochie River. There is a small population above the point where the water is taken. The river water is very turbid, containing much clayey matter in suspension, and it is therefore doubtful if any typhoid fever germs could survive after the water has been treated by the thorough filtration process, which leaves it very clear. It is definitely known that typhoid fever is not only propagated through public water supplies, but also in other ways. Milk has been shown to spread the disease, and there are still other ways in which it is done; therefore the speaker did not regard the specific arguments advanced by Mr. Hill on the typhoid fever death rate based only on the water supplies as very sound.

CORRESPONDENCE.

S. Bent Russell, M. Am. Soc. C. E.—The author states that the filter was only some weeks in operation. Judging from some years' experience with experimental filters, the writer would consider this time of some weeks altogether too short to show the practical working capacity of such a filter as the one described. Mr. Russell.

On examining the records of the experiments with the St. Louis filter described in the paper, it will be found that while the new filter started with a very low head, after some 20 reversals had been made, it took a foot of head to start it after cleaning. Again the head of starting after reversal increased steadily, and at the end of from 6 to 12 months the entire filtering material had to be removed. This showed that reversals of flow did not thoroughly clean the sand. Had the filter been run at the rate of 500 galls. per square foot per day instead of 50 galls., it would probably have been hopelessly clogged inside of a month. That is to say it would have been past relief from reversal of flow. Furthermore, if the sand in the Tacoma filter were not thoroughly cleaned by reversal, the oxide of iron used as a coagulant would have a strong tendency to cement the sand together so that it could not be used again, judging from the way the iron trimmings and punchings behaved.

The development of the American mechanical filter on the lines that it has followed indicates pretty plainly that filter sand cannot be cleaned properly by a simple flow of water through the interstices. The sand must be disturbed either by a rake of some kind, or by jets, or by a current strong enough to upheave and agitate the sand.

Returning to the coagulant used, the author's words would convey some doubt as to its success with a high rate of filtration. In his estimates he allows nothing for the scrap iron used. If enough iron were used to have much effect on the water the cost of it would be a considerable item, judging from the proportion of iron consumed in the Anderson process. Of course much would depend upon the quality of the water when it reached the filter, and there seems to be nothing very definite on that point in the paper.

To sum up the foregoing, in the judgment of the writer, the Tacoma filter would not prove a success in the purification of water containing much solid matter.

It is apparent on reading the paper that the author has experienced some of the difficulties that beset the path of the engineer who invades the labyrinth of the filter problem. He has displayed unusual courage in giving whatever data or lack of data he has obtained in his experience. The literature pertaining to mechanical filtration is most meager

Mr. Russell. indeed. This is remarkable when the sums of money now invested in mechanical filters in the United States are considered. Do the parties now making and using such filters wish to suppress the truth or are they ignorant of what the filters really are doing? It seems impossible to obtain authoritative and reliable data as to what a modern filter will do. Perhaps the main difficulty lies in the fact that the subject is much too intricate for the capacity of the ordinary manufacturer or water-works superintendent. If this be all, the large manufacturers should advance the necessary funds, and employ one or more engineers of standing to make efficient tests of filter plants now in operation and report the same for publication.

In view of the slighting treatment shown the mechanical filter by Mr. Allen Hazen in his new book on filtration, and in view of some recent reports by engineers favoring the European system, something of the sort ought to be done. The makers of filters should either prove their claims or cease to advance them. To be of value the tests would have to be made according to scientific rule, so that the results should be at least as definite as in the case of an engine or boiler trial.

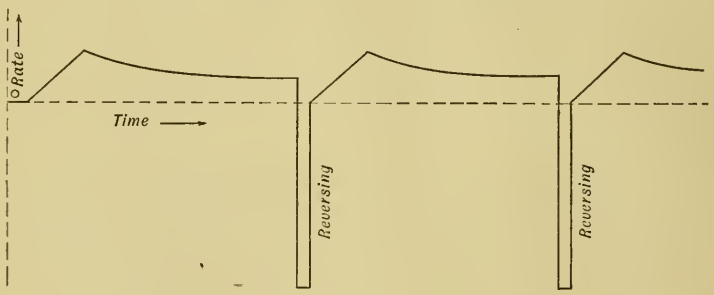


FIG. 5.

The writer for the last few years has had charge of some rather painstaking experiments on the purification of river water, and has at least learned something about the difficulties in the way of a scientific study of water filters. The primary factors in such an investigation are the water, the impurities or solids to be removed, the sand, the coagulant (if used), and time. As each factor is variable, the number of possible combinations approaches the infinite. No simple formula or curve diagram can be expected to represent the true relations of the factors. Take the case where the impurities, sand and coagulant are constant in their effect in a mechanical filter operated in the usual way and so as to give the maximum flow of filtered water. The filter must be started slowly or the filtrate will not be clear. Gradually the head and rate are increased until the maximum allowed head is reached; the maximum rate of filtration is also reached then. The head may now remain constant, while the rate will gradually decrease as the sand becomes clogged until the time for reversal comes around.

The rate of filtration may now be said to become negative, if clear water is used for washing. When the sand is clear another cycle is begun. Mr. Russell.

If a profile of the rate of filtration as it changes from hour to hour is plotted, it will have a form somewhat like Fig. 5. To get the capacity of the filter per day the average straight line must be found. The form of the diagram will change with the depth and firmness of the sand, with the proportion of coagulant used, with the purity of the water, and probably with the temperature of the water.

To obtain comparable results some means of recording the quality of the water and the quality of the sand must be adapted, and thanks to recent experiments this can be done with reasonable success. It is to be hoped that future experimenters will make some effort toward uniformity of methods. Until something better appears, all filter sand should be analyzed by the method described by Mr. Hazen.

EDMUND B. WESTON, M. Am. Soc. C. E.—As the thorough purification of water at the present time may be considered a science, and in order to form an intelligent opinion of different methods of water purification, it is eminently essential that reliable results which have been obtained by different processes should be given for comparison. It would, in the opinion of the writer, be a valuable addition to the information contained in the paper if the author would give the following data: (1) The percentage of the number of bacteria contained in the applied water that was removed by the process he mentions. (2) The percentage of albuminoid ammonia removed. (3) The percentage of ready-formed ammonia removed. (4) The percentage of color removed. (5) The percentage of the total amount of water filtered during a run that it was necessary to use in order to wash the filtering medium. (6) The percentage of the total amount of water filtered during a run that it was necessary to waste, after washing the filtering medium, before the filtered water arrived at its normal condition. Mr. Weston.

The following results, which are germane to the subject, are the average results derived from experiments which were conducted under the special direction of the writer in 1893 and 1894, at Providence, R. I., with an experimental gravity mechanical filter 30 ins. in diameter:

Water bacteria removed	98.6%
Applied *Bacillus prodigiosus* removed	99.8%
Albuminoid ammonia removed	70.0%
Ready-formed ammonia removed	91.0%
Color removed	72.0%
Percentage of the total amount of water filtered during a run necessary to wash the filter bed	4.9
Percentage of the total amount of water filtered during a run necessary to waste after starting the filter	2.9

About nine months were devoted to making the experiments, which included the warmest and coldest seasons of the year.

Mr. Weston. Basic sulphate of alumina was added to the applied water at an average rate of 0.6 grain per gallon of applied water. The average rate of filtration was 128 000 000 galls. per acre per 24 hours, or about 2 938 galls. per square foot per 24 hours. The average length of run of the experimental mechanical filter was 16.7 hours for a rise in height of 4 ft. of water after the filter commenced to discharge at an average rate of 128 000 000 galls. per acre per 24 hours.

The average results obtained from the many runs of the experimental filter covered a period during each run commencing from 20 to 30 minutes after the filter was started to the end of the run. The filtered water did not reach its normal condition until at the expiration of the 20 to 30 minutes mentioned, and was therefore allowed to run to waste.

Great pains were taken in order to ascertain if any of the applied basic sulphate of alumina came through the filter in the filtered water, and the writer is of the opinion that it was satisfactorily demonstrated that none passed through the filter in its original state, although a minute quantity of finely suspended aluminic hydrate could be detected in the effluent water by the application of the logwood test.

Mr. Hill. John W. Hill, M. Am. Soc. C. E.—Apart from the statement that the filter was operated at rates of 500 to 1 000 galls. per day per square foot, no information is given of its efficiency. The true test of a filter, in the light of the present knowledge of such matters, is the removal of bacteria, and any filter which will satisfy this requirement will probably satisfy all others. If any sanitary advantages were obtained from the filtration of water by the method described in the paper, it is to be regretted that these were not recorded with the same care as to detail which the author gives to the items of cost.

Certainly the cost of construction when the capacity of the filter is considered was very low, and the cost of operating a plant with a capacity of 5 000 000 galls. per day is also very low, but if this cost is attended only with an incomplete clarification of the water, and the removal of some of the comparatively coarse algæ, which might be done with a much simpler apparatus, it is doubtful if even this small cost of $1 40 per 1 000 000 galls. of water treated is justified. The writer has contended* that a partial purification of water by these high-speed and some of the low-speed filters is more dangerous from a sanitary standpoint than no attempt at all at purification, because the consumer, after the water passes any filter, takes its wholesomeness for granted, while in many cases on record no improvement at all has been made in the potability of the water, although all objectionable color and odor may have been eliminated from it.

Gauging the efficiency of filters by the typhoid fever death rates of the cities supplied, the most successful results have been obtained

* See the *Transactions* of the American Society of Civil Engineers, Vol. XXXII, p. 130.

from the fine sand filters in Holland, and these are operated at low rates per square foot of filter area. The effect of careful filtration of the Elbe water for Hamburg, under the direction of an expert like Dr. Dunbar, is well shown in a comparison of the typhoid fever death rates of this city for the years 1892 and 1894, during the former of which the raw river water alone was used, with a death rate of 34 per 100 000 of population, while during the latter year filtered water alone was used, with a typhoid death rate of six per 100 000 population, or the reduction of typhoid fatalities was nearly 82 per cent. Mr. Hill.

From a compilation of statistics upon which the writer is now engaged, some facts with reference to the practical results of mechanical filtration will be of interest in this connection. Of three American cities using water from mechanical filters, the following statistics are given:

Death Rate from Typhoid Fever per 100 000 of Population.

	1890.	1891.	1892.	1893.	1894.	Average.
Davenport, Ia	19	11	35	17	26	21.5
Knoxville, Tenn	101	45	37	67	59	62
Atlanta, Ga	149	119	87	66	43	93

Now take several cities abroad using fine sand filters, with low rates of filtration per square foot of filter area, for the same years:

	1890.	1891.	1892.	1893.	1894.	Average.
The Hague	3	12	4	2	3.4	5
Berlin	9	10	8	9	4	8
London	16	15	11	16	15	15

Of all cities in the United States using mechanical filters, from which the writer has been able to obtain the vital statistics, Davenport, Ia., shows the lowest typhoid fever rate, but even this city for the past five years has had an average rate four times that of The Hague, two and one-half times that of Berlin, and 40% greater than that of densely populated London, where 83% of the water supply is obtained from three streams which would scarcely attain the dignity of rivers in this country. But all the water for London not drawn from the deep wells in the Kent chalk is carefully filtered under government supervision, and at low rates per square foot of filter area. The London filters are usually worked at rates of 37 to 40 galls. per square foot of filter area per day, less than one-twelfth of the rate for which the Tacoma filter was planned, and one-twenty-fifth of the rate at which the paper states it was occasionally worked.

High speed in any filter can be obtained by the use of thin beds of coarse sand, but high efficiency is more to be sought than high speed,

Mr. Hill. and, judging from the present information upon the subject, this is to be had only with thick beds of fine sand operated under low heads and at low rates per square foot of filter area. A test of a filter should show the changes effected in the quantity and character of the suspended and dissolved organic matter, and in the bacterial condition of the water, and especially the influence of the filtered water on the typhoid fever death rate of the community supplied.

While the most satisfactory showing in the typhoid fever rates has been obtained from a city depending upon filtered water, The Hague, still among the principal cities of the world showing the highest death rates from typhoid fever are several using only filtered water; Dublin, for instance, with an average rate for the past five years of nearly 60, and Atlanta, with an average rate of 93, per 100 000 of population. As a rule, cities like Vienna, Munich, New York, Boston and Newark, which seek their water beyond the reach of serious pollution, show much better continuous and average rates of typhoid fever than the cities which depend upon previously polluted unfiltered or filtered water, which seems to indicate that safety in the matter of public water supplies is in the direction of sources of natural purity, rather than in attempts to purify a polluted water by filtration, at least by the high-speed filtration as described in the paper.

Mr. Hazen. Allen Hazen, Esq.—The question of water filtration is an important one in the United States where, as a matter of necessity, many cities take their water supplies from rivers and lakes which are muddy and which are polluted by sewage. Processes of filtration have been developed in densely populated European countries for purifying the water from such sources, which leave little to be desired in the results obtained. The cost of these processes, however, appears to be excessive to American communities which have obtained their water without other costs than that of pumping, and do not, as yet, realize the great value of a pure water supply.

As a result of this condition, there are numerous and determined efforts to devise and secure the adoption of other and more rapid processes of filtration by which water can be treated at less expense than by the comparatively slow and thorough processes commonly employed in Europe. The one all-important question to be raised in regard to all processes of this nature is whether or not they are capable of purifying the water in such a way as to make it suitable for the purposes for which it is intended.

The paper describes a filter which was constructed at a very small expense and which allowed a considerable quantity of water to pass through it. The author has unfortunately neglected to state what degree of purification was obtained, whether or not the water was rendered suitable for city supply, and whether or not the filter could be depended upon to remove the algæ or the bacteria and other organisms which were or might have been present in the unfiltered water. The

paper thus has only a suggestive value, and while ideas may be obtained from it which will be useful in designing other filters, it will not be safe in other cases to follow the construction described without investigating the results obtained by this apparatus, which are not given in the paper. Mr. Hazen.

In regard to the details of the paper, the rate of filtration is given in the somewhat unusual unit of gallons per square foot daily. It is greatly to be desired that some unit for rate of filtration should be adopted which is applicable to all the rates ranging from quantities used in sewage treatment by irrigation to the much higher quantities used in water filtration by European methods, and to the still very much higher rates employed in mechanical filters for treating water. At the present time the quantities are expressed, according to circumstances, in English or American gallons per square foot, per square yard or per acre, or in vertical inches or feet, and for time, intervals of minutes, hours or days, making a total of 24 possible combinations, the greater part of which have been used in some way or other in connection with filtration. There is probably no other unit so convenient as the metric one of a vertical column of water equivalent to 1 m. daily on the effective filter surface. But as it is impracticable for various reasons to use this unit in the United States at the present time, the unit of 1 000 000 galls. per acre daily, which is very nearly equal to the above-mentioned metric unit, has many advantages, notwithstanding the objection that may be raised to a unit of area as large as an acre for areas as small or smaller than that of the filter described in this article.

In regard to covering the filter which is spoken of as "an absolute necessity in this climate," that is, at Tacoma, the writer's information is, perhaps, less complete than could be desired on that climate, but from statistics at his disposal the winter temperatures of the Pacific coast are very much higher than those of the Atlantic coast points and correspond more nearly with those of western Europe. There open filters are successfully used in much higher latitudes than that of Tacoma, and it may be a question whether covering would be necessary or desirable in that place.

The sand used seems to have been rather coarse; it is stated that in order to allow the high velocity, 500 galls. per square foot per day or 21 780 000 galls. per acre daily, it was necessary to use "a sand of large grain, free from all finer matter, and there was the danger that the chief usefulness of a filter, viz., its ability to thoroughly strain out all impurities, might be lost." This is, indeed, a serious danger, for the efficiency of filters decreases rapidly with the size of the sand grains employed. To avoid this it is stated that a coagulant was used. The coagulant selected was metallic iron, over and through which the water was taken, but it is very doubtful whether the term coagulant should be extended to cover metallic iron used in this way. A coagu-

Mr. Hazen. lant in filtration is strictly a substance added to a water or liquid in solution, which, by chemical or other changes, becomes precipitated in the liquid and forms an insoluble substance which surrounds and combines with suspended or dissolved matters in solution, and allows the whole mass to be more readily removed than the matters could be removed by themselves. If water is agitated in contact with metallic iron, it will sometimes (depending upon the composition of the water and other circumstances) take a portion of the iron into solution as ferrous carbonate, and this may subsequently become oxidized and precipitated as ferric hydrate. In case enough iron is dissolved to form a substantial precipitate, and other conditions are present to make it do so, the iron may be regarded as a coagulant, but in the present instance there is no reason to think that any such quantity of iron went into solution or was precipitated; in fact, the faint milky tinge mentioned shows that this was not the case, and the iron employed was only useful in so far as it acted as a screen to remove insects and other floating matter in the water, as mentioned by the author, and as these would have been removed in any case by the sand, it is not apparent that the iron aided the process in any way.

Mr. Cummings. Robert A. Cummings, Assoc. M. Am. Soc. C. E.—Criticism of this paper is practically disarmed through the author's admission that it is not in strict accordance with the best ideas. Since he does not give the results of filtration by his new form of filter, it may be of doubtful efficiency. Indeed, it is not clear how such a filter bed can be efficient.

The author appears to have concluded that the action of sand filtration is purely mechanical. The successful filtration of water depends on the sound appreciation of this action. It is generally understood that there are two distinct actions in sand filtration, viz., the straining and the adhesive actions. On the one hand, water holding matter in suspension is strained as through a sieve; and, on the other hand, water is filtered through a sand filter as in passing through a settling basin. The action in the former case may be considered as a retaining one, and admits of a high velocity; and in the latter as an adhesive one, requiring a low velocity for the settlement of suspended matter. The attraction by and to the sand grains of particles so much smaller than the interstitial spaces substantiates the correctness of this last view. It is now well known that when the usual limit of rate of flow is exceeded, the efficiency of filtration is depreciated. All methods of simple sand filtration at extraordinary velocities of flow cannot be relied upon until it has been clearly shown that they are efficient. For the removal of bacteria, sand filtration undoubtedly depends upon the delicate deposit of organized matter which forms on the top of the sand. The efficiency will depend upon the quality of the water, the temperature, the ripeness of the sand, etc.

The advantage claimed for upward flow filtration is the high velocity

obtainable. This can be secured only at the expense of efficiency of filtration. It has been pointed out that with a high velocity the action is that of a mechanical strainer, which retains only the coarser particles, and has a very small bacterial efficiency. Does the film so necessary in sand filtration form at all in the upward flow filters? The writer thinks not. If it does, at what point? Suppose it forms in the sand at the top of the filter, for it will not form in the gravel; the whole filter bed becomes impregnated with organic matter and requires renewal at each clogging of the beds. The simple reversal of flow of filtered water will not cleanse the bed. The whole mass of sand must be cleaned and exposed to light and air. The waste of filtered water for this purpose is a serious objection, and the pollution of the streams below is not permissible. Mr. Cummings.

The writer was recently consulted with regard to much trouble with the upward flow filters at Wayne, Pa., which were found washing the sand, or rather agitating the sand, as noted by the author. The filters had a bed composed of crushed slag, gravel and sand. They are not in use now.

It has been found at the Berlin Water-Works that open filter-beds are more efficient than covered ones, and the late Mr. Gill, chief engineer, has stated that the choking of the pores of the filter and the growth of algæ at the surface of the sand are most valuable aids to the micro-biological improvement of the water.

It is agreeable to note that the result of the author's experience with iron as a purifier should be in accordance with results found elsewhere. It may be noticed, however, that Mr. William Anderson used spongy iron at the Antwerp Water-Works several years ago, and it purified the water satisfactorily, although the spongy iron soon became choked. After many experiments the latter was replaced by the agitation of metallic iron with the water, and by sand filter beds.

The apparatus for the Anderson process may be briefly described as consisting of a revolving purifier, a small engine for revolving the purifier and a tank fitted with a fine screen for separating coarse particles. The revolving purifier is a wrought-iron cylinder supported longitudinally on hollow trunnions fitted with stuffing-boxes through which the inlet and outlet pipes pass. The inside of the cylinder has curved ledges suitably arranged for scooping up the iron and showering it down through the water. Experience shows that the iron has very little tendency to move along the cylinder with the current; however, means are provided to check it. The screen is to intercept algæ and other large impurities, that may be in the water before it reaches the purifier, from the sand filter beds. The growth of algæ never takes place after the water has been purified.

The following explanation of the action that takes place with the agitation of metallic iron in water containing organic matter is given by Dr. E. Divers:

Mr. Cummings. "Water containing carbonic acid dissolves iron in the form of carbonate, at the same time liberating hydrogen when no deoxidizable matters are present. One effect, well known, of depriving water of its dissolved carbonic acid, is to precipitate any calcium, magnesium and iron carbonates which the carbonic acid may have been keeping in solution. In this way spongy iron will become coated over and cemented together into a hard mass by some calcareous waters. What iron carbonate is not thus precipitated becomes afterwards changed by atmospheric oxygen into ferric hydroxide and deposited as a rusty sediment. Oxygen, nitrite, nitrate and no doubt some other of the oxygenous matters in solution in the water are consumed, partly by contact with the metallic iron in presence of the carbonic acid and partly afterwards by the iron dissolved as carbonate.

"It is not easy to admit that the action of iron upon water holding carbonic acid in it can be effective in destroying or precipitating dissolved albuminoid matters from the water. Rather must it be considered that the purification of water from matters indefinitely classed as albuminoids consists in the removal of organisms living in the water. The activity of iron in depriving water of bacteria is just what might be expected, since a microbe in water holding carbonic acid gets into a field of powerful chemical action when it comes in contact with bright iron and will naturally be paralyzed, if not killed outright. Further, water, freed as it is by iron of both carbonic acid and oxygen, must be ill-fitted for bacterial life, while the dissolved ferrous carbonate will, even in the minute quantity of it present, be a powerfully poisonous agent, like all such reducing or deoxidizing agents.

"In accordance with this account of the probable mode of action of the iron in killing and removing bacteria is the fact that it is only quickly after contact of the water with the iron that bacteria are so strikingly diminished in number, for towards the de-aerated water the iron becomes passive as glass, and new generations of bacteria may knock up against it with impunity, if otherwise able to live in the water."

Having considered the *modus operandi* of the Anderson process, it will be readily appreciated that the author's method of using old horse shoes, etc., added little or no value toward purifying the water.

Dr. E. Frankland has recently stated that a filter ought to be cleaned as seldom as possible, owing to its inefficiency for some time after cleaning, and he recommends an increase of storage capacity, which would give the sand filter less work to do; but this would only lengthen the time for the ripening of the sand. In the Anderson process an inorganic film of ferric oxide is deposited on the surface of the sand from the time the filter is started to work, which appears to obviate this difficulty. The conversion of the protosalt of iron formed in the water during its passage through the revolving purifiers into insoluble ferric oxide results in a coagulated precipitate, in which bacteria are entangled and imprisoned, the coarse-grained deposit thus formed being readily arrested at the surface of the sand, into which it does not penetrate. About four hours is given for the formation of the film.

From the analysis by Dr. Van Ermengen of a series of samples

taken hourly from a sand filter outlet at the Antwerp Water-Works, on September 9th, 1893, the folowing results are given: Mr. Cummings.

Hours after starting filter.	Number of microbes per cubic centimeter	Hours after starting filter.	Number of microbes per cubic centimeter.
4	30	11	30
5	30	12	20
6	27	15	50
7	320*	18	52
8	25	24	25
9	30	30	25
10	39		

In the report of Dr. Henry Leffman and Dr. William Beam on the results of a six-month trial at the Belmont Water-Works, Philadelphia, the above results are fully confirmed. Their report reads:

"They show that it acts to remove suspended matter of every character, and to diminish ammonium compounds, dissolved organic matter, whether nitrogenous or not, nitrites or nitrates. The reduction in the number of microbes is especially to be noted, not only so far as the actual figures are concerned, but in view of the fact that there is now no doubt that the dangerous effects of water are due to these agents."

In the annual report of Dr. P. Miquel, published in the *Annuaire de l'Observatoire de Montsouris* for 1894, he states that the average of 22 samples shows that the number of bacteria in the original water was reduced 99.57% after purification and filtration, five showing a percentage of improvement of 99.9 per cent. The average number of bacteria in the original Seine water during the period of the test was 396 000, and the average speed of filtration was 75 galls. per square foot per 24 hours.

A striking example of the difference in the efficiency of a sand filter in the chemical improvement of a river water when used with and without the Anderson process is furnished by the analyses by Dr. H. Swete. In a paper read before the Sanitary Congress at Portsmouth, England, September, 1892, he states:

"I have divided these analyses into three stages: *First.*—The period before the purifiers were set to work. *Second.*—The intermittent period, during which period the continual structural alterations were being made to the filters, causing disturbance to the sand, and the crust formed on the surface of the filters. *Third.*—The third period, when the works were completed and a fair average result of what the system will effect could be estimated. These periods show the following percentage improvements in three important points:"

* Probably due to an accident to the sample.

Mr. Cummings.

PERCENTAGE OF IMPROVEMENT.

DATES.	Free ammonia.	Albuminoid ammonia.	Oxygen to oxidize organic matter.
First period	45	30	35
Second period	77	60	64
Third period	100	70	66
River in flood	97.8	75	73.5

It is now generally admitted that ordinary sand filters require the greatest care to avoid sudden variations in the rate of filtration, which destroy their delicate film. The ferric oxide film is not nearly so delicate, and allows greater variations in the rate of filtration; hence a greater speed of filtration can be obtained with as satisfactory results as from the larger area required for ordinary sand filters.

In the spring of 1892, experiments were made at Hampton, near London, to determine whether the life of a sand filter, when used in conjunction with a revolving purifier, would be lengthened or shortened by the addition to the water of the iron oxide. Two small sand filters, each having an area of 36 sq. ft. of sand surface, were constructed side by side, the one receiving Thames water direct from the river, and the other the same water passed through a purifier delivering at the rate of 500 galls. per minute. Owing to the shallowness of the filter tanks it was only possible to maintain a depth of 1 ft. 6 ins. over the sand in the filters, a circumstance unfavorable to the Anderson process as the time for aeration and coagulation of the iron oxide was reduced to less than half the normal period allowed.

The results of the trials showed, however, that the filter receiving the water purified by iron delivered from 25 to 50% more in the same number of hours than the filter working under ordinary conditions, the greatest difference in favor of the purified water being when the river water was most turbid.

The writer is informed that the Compagnie Génèrale des Eaux and the Compagnie des Eaux de la Banlieue, supplying water to Paris, have recently installed large plants for the purification of the Seine water by the Anderson process.

In a paper on the "Revolving Purifier Process," read by Mr. William Anderson, at the meeting of the British Association at Cardiff, in 1891, the author states:

"With regard to the working expenses, accurate figures can be given of the cost of the process during the year 1889, at the Dordrecht Water-Works. The purifier at these works has a 14-in. pipe and can purify 1 250 000 galls. daily; it is automatic, being driven by a small water motor worked by the water which passes through the cylinder, a very economical arrangement.

"The figures for the year 1889, are as follows:

Iron consumed per 1 000 000 galls. treated ..	13½ lbs.
Speed of filtration per square foot per 24 hours	100 galls.
Total quantity of water treated during year...	194 000 000 "
Total cost for the year, including iron filter cleaning, maintenance and repairs.....	$267.50
Average cost per 1 000 000 galls. purified ...	1.37

Mr. Cummings.

"The iron used is in the form of punchings, ⅜ in. to ¾ in. diameter, costing $11 50 per ton."

More recent figures from the small plant at Boulogne-sur-Seine give an average cost of 0.0016 franc per cubic meter, or about ⅛ cent per 1 000 galls. ($1.43 per 1 000 000 galls. for a small plant of 1 100 000 galls. per day).

Mr. Hawks.

A. McL. Hawks, Jun. Am. Soc. C. E.—It must be borne in mind that the plant described in the paper was constructed in great haste and for a very small sum to remove some specific causes of complaint from the water then being supplied to the city of Tacoma. The work could not be delayed long enough to send samples of the water to a biologist and carry on correspondence sufficient to determine the exact causes and best means of obtaining entire relief from all sorts of impurities. It was a case which demanded immediate action. That no analyses could be given is a matter of regret to the author; samples for this purpose were taken and were furnished to the company, but no results were ever given, if analyses were made. The actual result of the operation of this plant was the removal of the causes so that no further complaints were made.

Moreover, the work was commercially successful, which is the final test of all engineering work, for though engineers may plan ever so skilfully and scientifically, if they do not gain the approval of the ordinary business man their work is of little account. The plan of the Tacoma filters was laid before the Water Commissioners of the city of Wilmington, Del., who were then considering filtration of the water supplied to that city. Two of the members made a trip to Tacoma for the purpose of investigating this plant, and spent several days on their mission. They visited the filters with the author, and had them blown off and refilled and blown off a second time. Even though the beds had been indifferently cared for during nearly five months after the author had turned them over to the operating department, the visitors were highly pleased with the working of the plant. They afterwards visited the officers of the water company and made private inquiries for the purpose of determining the improvement in the character of the water after the filters were put in service. So well were the commissioners satisfied with the result of all their inquiries, that they returned to Wilmington and authorized the construction of a large plant for their city, introducing all the essential features of the author's plant and the additions which had been advocated by him but elided in the effort for greater economy of con-

Mr. Hawks. struction and operation. The Wilmington plant has been constructed about two years, and should be able to furnish many items of interest and give the very data which are missing from the paper. It was hoped that such items and data would be drawn out in discussion.

No one in these days thinks "the action of sand filtration is purely mechanical." It is distinctly stated that the delicate deposit mentioned by Mr. Cummings was formed at the bottom of the sand layer in the glass-sided upward filter at St. Louis.

The samples of water taken from the blow-off of the Tacoma plant when the filters were reversed showed a large amount of filth was being removed from the water and carried entirely without the beds; if a small amount was caught and held by the gravel, it would only serve to form the film upon the surface of the sand so necessary for the best results. So long as the sand is not permitted to boil, so long as it is perfectly quiescent, this film does not penetrate the surface any appreciable distance. The amount of filtered water wasted in reversal is 0.5% of the filter's daily capacity. Air is thoroughly drawn into the beds by the suction of the water subsiding at the time of reversal. There can be no reason for letting light and air in upon the top of these filters; the water standing there has already been purified. It would be like purifying water in an ordinary filter, and then exposing the clear-water basin to fresh contamination.

The iron filings, turnings, punchings, etc., began to clog within three days after they were placed in the flume; the smaller particles adhered to the larger ones in the first week, and in two weeks there were large coherent masses of rusted iron in the flume. Is it not possible that what has hitherto been unexplained, viz., the myriads of minute insects collected on the iron in the flume may be due to some such action as that described by Dr. E. Divers in the correspondence by Mr. Cummings? While they were in the water no movement could be seen, but after they had been exposed to the air for a few minutes, say from five to ten, they would begin to move, and, as the water evaporated, would become very lively. Why is "bright iron" specified? If the iron was removed and subjected to fire frequently enough to permit it to give off the protosalt when again immersed, is not that sufficient?

It is said with regard to the Dordrecht Water-Works: "The purifier * * * can purify 1 250 000 galls. daily," while a little farther on it is stated with regard to the operations during 1889: "Total quantity of water treated during the year, 194 000 000 galls." This may mean either that the purifier ran half the time or that it ran at half the "speed of filtration per square foot per 24 hours, 100 galls." If the sum of $15.06, the cost of the iron for 194 000 000 galls., be deducted from $267.50, the total cost for the year, it leaves $252.44 for filter cleaning, maintenance and repairs. As the apparatus is said to operate automatically, no part of this sum can be for operation.

Mr. Hawks.

The filter was run by the author but a few weeks. It was operated some months after that by the company, but without any records being kept. It is true the St. Louis upward filter clogged steadily, though reversed frequently, and needed greater starting head after each reversal ; but this filter only permitted the water to be drawn off comparatively slowly. Once, through accident, it was very suddenly emptied, with the result that it did not require any greatly increased head to start it again.

The coagulant in the Tacoma plant did not have the effect of binding the grains of sand together. For the purpose of determining that fact, and also as to the penetration of foul matter into the bed as well as could be detected by the eye (the greater part of the filth had a distinctly visible light brown color), a small pit was dug in the middle of one of the compartments after the plant had been running about a month. The sand was perfectly free from any such binding, and appeared to be perfectly clean to the surface of the gravel beneath. It was intended to keep an account of the iron consumed, but the time during which records were kept was not sufficiently long to show any appreciable loss upon so great an amount as 8 500 lbs. At the rate given for the Anderson process in the Dordrecht works, the loss for a month in this plant would not have been 10 per cent. It was intended to weigh the iron every three months to determine the loss, but this was never done.

The author agrees with Mr. Hazen that some unit for rate of filtration should be adopted ; but why not that of gallons per square foot ? It is much easier to use than millions per acre, and that it is not unusual is shown by three of the discussers of the paper adopting that unit, and one using it in connection with the 1 000 000 galls. per acre unit.

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No. 772.

NOTES UPON THE CONSTRUCTION OF A WATER SYSTEM FOR PLACER MINING, AND SUGGESTIONS FOR A NEW METHOD OF DAM BUILDING.

By ROBERT BREWSTER STANTON, M. Am. Soc. C. E.

PRESENTED NOVEMBER 6TH, 1895.

WITH DISCUSSION.

The experience gained in overcoming even small difficulties and the deductions drawn from successful undertakings, even though in themselves of minor importance, may be of service to other engineers; hence these notes.

In the spring of 1893 the author began the construction of a pipe line, reservoirs, etc., for the opening and development of a placer mine on the summit of the Coast Range in southern California. The property, consisting of some fourteen claims, is situated on the eastern slope of San Antonio Peak, generally known as Baldy Mountain, in San Bernardino County, California, and lies at an elevation of between 7 500 and 8 000 ft. above sea level, in the San Gabriel Mountains. The

rich placer gravel is found in the well defined channel of an ancient river bed, which was elevated as the mountains rose, and left high and dry upon the summit of the ridge. It is not the intention to go into the geology of this region, nor into its ancient history, except so far as it bears upon the engineering features of the work.

The general course of the old river, at the point under consideration, was nearly southwest. Coming as it did from the north or northeast, it left, with perhaps other gold-bearing streams, deposits of auriferous gravel scattered over the Mojave Desert, and had its mouth some 10 to 15 miles south of San Antonio Peak, when the great San Gabriel Valley, in which is the town of Pasadena, and also the country around Los Angeles, were still under the waters of the ocean. Whether these southern rivers were the lower portions, and whether here were situated the mouths of part of the great ancient river system of the North, which has produced so much placer gold, it is not necessary for present purposes to consider. Yet it is interesting to observe some of the well defined river mouths on the southwest slopes of the ranges in southern California, where gold-bearing gravel has been deposited over thousands of acres, to a depth of from 300 to 500 ft., in well defined ocean bays. This gravel is mixed with all kinds of marine shells, some immense oyster beds in these deposits now lying from 2 000 to 3 000 ft. above the present sea level, and it may not be out of place to note, as has been demonstrated from well authenticated documents, that the first discovery of gold in California was made in one of these bay deposits, that of the San Feliciana, in Los Angeles County, about 40 miles north of the city of Los Angeles, by Francisco Lopez, from six to sixteen years before the much glorified Marshall made his discovery in 1848 at Sutter's Mill.

Topographical Features.—The topography of that portion of the ridge on which the old channel was raised has almost entirely changed during the ages since the upheavals. The elements have worn away the greater part of the old channel. There are recent gulches cut across and parallel with the old stream, and most of the country surrounding its course lies below the present level of its bed-rock. In three places the old bed is left. One is where it passes through a gap between two hills, and both banks of the river are left with the gravel some 80 ft. deep between them. In another but one bank and a portion of the bottom rock remains on the steep side of the mountain.

The third is a level piece of the bed-rock, with deep gulches on every side, and this capped with a cone of gold-bearing gravel; while in the long gulch, that has cut diagonally across the old river, has been formed a secondary deposit of gravel and gold, washed down from the channel above.

The Water Supply.—Although other things are required, the one essential for the operation of placer mining is plenty of water. While, as remarked, the gravel was left high and dry, a water supply had been provided by Nature about 2.5 miles distant under the summit of San Antonio Peak. On the south side of the range, at some previous time, the side of this mountain slid down toward the sea, leaving a tipped-up basin with apparently a solid rock bottom and with high mountain walls on all sides save one, and covering an area of some 1 200 to 1 500 acres, the lower edge of the basin being about 2 200 ft. below the summit of the mountain. This basin has been gradually filled with débris from the mountain above, consisting of broken rock and earth, and part of its surface is now covered with heavy timber.

Although within sight of the orange groves of Pomona and Ontario, some 15 miles distant, in the valley below, these mountains are covered over six months in the year with heavy banks of snow. The weather is not severely cold during the whole of the winter, yet at an elevation of 8 000 ft. the author has experienced storms of snow and wind as severe and almost as cold as any in the higher elevations of the Rocky Mountains or in the Dakotas. Into this great basin, the snows drift and pack during the winter, and even under a southern California sun remain until late in July. At the lower edge of the basin, at the head of San Antonio Creek, burst out springs, which form the source of the water supply for the mines.

The Reservoirs and Pipe Line.—The water from these springs flows over the bed-rock at the lower edge of the basin, drops into the gulch at the head of San Antonio Creek, and sinks into the débris that fills the gorge. To catch this water and turn it into the head of the pipe line a small reservoir or catch basin was built.

The débris was simply leveled off across the gulch, about 60 ft. wide, leaving a bottom of fine broken granite rock, with the rock walls of the hill on three sides, two of these sides being slate and the upper or back granite. On one side this bottom was sunk 3 or 4 ft. deeper for a waste and cleaning gate.

At the lower or outer side of this collecting reservoir were two large granite bowlders. From a notch cut in the slate wall a pine log dam 6 ft. high was built across to the bowlders and to the opposite wall, lined with a double thickness of plank sunk into a trench in the débris, and the fine broken rock filled back against the plank. A waste gate was left through the dam at the lower corner opposite the pipe entrance.

In considering the methods of construction of this whole work, the fact must be borne in mind that the body of gravel to be hydraulicked, while rich in gold per cubic yard, is of limited extent, and will all be washed out and the works be of no value in a very few years, and hence plans and methods were adopted for some parts of the work not suited to more permanent structures. The head of the pipe was run through the planking between the logs, with a strainer on the end to keep out leaves and sticks.

It will be seen that in this small basin there were five kinds of material to be variously joined together by a water-tight lining; shattered slate to be joined to the solid granite and to the plank dam; the bottom of finely broken stone, after being well rammed, to be covered and joined water-tight to slate, granite and planking, and the planking to be joined to the iron pipe. This was done by covering the whole inner surface with two coats of thoroughly prepared asphalt paste, applied hot. After the small holes, cracks and crevices were filled, the thickness of the asphaltum was not over 0 5 in.

This asphalt paste was made of 4 parts of the very best refined California asphaltum and 1 part of crude petroleum, without sand, boiled together until when cool it formed a tough rubber-like paste, brittle under the hammer when absolutely cold, and yet elastic and pliable with the least warmth, adapting itself to any slight change in foundation or walls. A tough rubber-like pouch resting inside a frame of rock and timber was thus formed. With this lining a perfect joining was made between slate and granite, timber and rock, gravel and plank, plank and iron, and, after two summers and one severe winter, is in good condition and well adapted for the uses intended, the only repairs being the addition of a few quarts of asphalt last spring. Further reference to this work will be made in the description of the pressure reservoir.

The Pipe Line.—The distance from the collecting reservoir to the pressure reservoir or penstock, just 2 miles, was along a very steep, rough and crooked mountain side. It would have been impracticable

to maintain a ditch or flume on account of the snow and rock slides from the mountains above. For this reason, and also on account of the nature of the material along the route, a pipe line was laid. A bench from 2.5 to 4 ft. wide was cut along the mountain side, and in this, next to the wall where the material was earth or débris, was sunk a trench in which was laid the pipe, but along the cliffs and in solid rock the whole bench was cut out to the bottom level of the trench. For a distance of some 200 ft. at one point, a half tunnel was cut in the face of a perpendicular cliff.

One feature of this work is worthy of note, as causing extra expense in construction. On account of the rock and snow slides from above, it was necessary to cover completely this pipe for protection, and owing to the very steep mountain sides almost every particle of the material excavated was lost in the ravines below. Hence to cover the pipe new material had to be excavated, and this, especially along the rock faces, increased considerably the expense of the work.

At three points the pipe is exposed in carrying it across steep and narrow gulches, this plan being considered cheaper at these places even if the line had to be replaced, and extra pipe was provided and left on the ground for that purpose. Across these gulches were built simple lattice spans 32 and 64 ft. in length, resting on trestle bent abutments. The spans were built of rough mountain pine lumber, the top and bottom chords being composed of one line of 2 x 12-in. plank faced with two lines of 1 x 12-in. boards breaking joints, and latticed and braced with 1 x 6-in. boards all simply spiked together with wire nails. The 64-ft. span was 6 ft. in height and 4.5 ft. in width, all of the material like the pipe, being transported for nearly 2 miles on men's backs, as is described a little farther on. This method of construction, though rough and in some respects unscientific, has for two years proved entirely satisfactory for the purposes intended.

The pipe laid was the ordinary California No. 16 gauge sheet iron riveted pipe boiled in asphalt, with driven joints. It was laid on a grade of 40 ft. per mile for the first 500 ft., the remainder being on a grade of 16 ft. per mile, except the last 1 500 ft., which was 26.4 ft. per mile.

The first 1 800 ft. of pipe was 12 ins. in diameter, 2 000 ft. was 10 ins., and the balance was 8 ins. in diameter. Cross and waste valves were provided at the upper and lower ends and at the head of the 8-in.

pipe. Air valves were placed about 1 200 ft. apart along the whole length of the line. After careful examination of the gulches to be crossed, etc., it was estimated that the pipe could be laid more cheaply on such a continuous grade, and it could be completely emptied of all water in winter without the use of so many expensive valves as would have been necessary if the gulches had been crossed by descending into them. It may here be remarked that the size of this pipe did not meet the approval of the author on account of the fact that, although it carries all the water coming from the springs in August, September and October, it allows about two-thirds of the water flowing in April, May and June to go to waste. The capacity of the pipe should have been such as to carry nearly all of this early melting snow, so as to work the mines at the greatest advantage and to the full capacity of the water during the early months, thus getting maximum results for the whole year. But he was overruled by the treasurer of the company on account of first cost.

The pipe, valves, material for laying the pipe, timber for dam and bridges, and asphaltum for reservoirs were all delivered by wagon at the lower end of the pipe line. In order to distribute this pipe along the line, and the material for the small reservoir and bridges, resort was had to primitive means of transportation. The weight of all this material, including the pipe from the penstock to the mines, was approximately 100 tons. This 100 tons had to be carried on men's shoulders to an average distance of 1.25 miles along a narrow path on this steep mountain side and around cliffs where a mis-step would have dashed men and load upon the rocks hundreds of feet below.

Although the author has had some experience with the French Canadian voyageurs in the pathless forests north of the Canadian Pacific Railroad, and in portaging heavy supplies around portions of western rivers, it had never fallen to his lot to experience the difficulties or expense of transporting in this primitive manner 100 tons of bulky material even the short distance of 2.5 miles, especially over such a steep side hill trail. When the time drew near to begin this part of the work the grading of the line was about three-quarters done and it would have been inconvenient to lose the grading force. There came near being a rebellion among the men upon this work, for the ordinary grader objected to being turned into a pack mule.

One of the foremen was sent down to secure the aid of some Mexi-

can Indian packers. A dozen of them were brought up and set to work. They worked all of one day and then, with two exceptions, vanished down the mountain during the next night without asking for their day's pay. A second attempt was more successful, and a larger band of Mexicans was secured, that remained to complete the work. After several days' trial, with a young engineer who spoke Spanish as a foreman, it was concluded that they were doing good work when two men carrying one piece of pipe between them made an aggregate of 16 miles per day over such a trail, that is, 8 miles out loaded and 8 miles back empty. The largest pieces of pipe weighed 210 lbs. each. It was thought then, to save the expense of the foreman, to set these packers a task, a custom followed largely in handling Mexican workmen. Sixteen miles a day was taken as a basis. The next day they began with this task and they were comfortably seated around their camp fires by three o'clock in the afternoon, working in this way. Two of the men would take between them a piece of pipe weighing 210 lbs. and trot along from 0.5 to 0.75 mile without stopping. The heaviest work was packing the 12-in. cast-iron gates to the outer reservoir. After taking them apart, the largest pieces weighed 425 lbs.; these had to be carried in the same manner over the same trail the full 2 miles. Four men went with each piece at one time. Two of them would carry it swung between them on a pole on their shoulders from 500 to 1 000 ft. according to the nature of the trail, and then give it to the other two, thus alternately carrying it the 2 miles.

It is rather a remarkable fact that on the Pacific Coast where so much of this light sheet-iron pipe is used, that manufacturers have not made a much lighter water gate to go with it. Nowhere among the large pipe manufacturers in San Francisco could anything be found for water gates except the ordinary cast-iron gates used with heavy cast-iron pipe.

One of the most difficult parts of the work in laying this pipe was where the line passed over an immense rock slide that came from 1 000 ft. above the line and extended as many more below. The top 10 to 15 ft. of the slide was the only portion that kept gradually sliding. After moving hundreds of tons of this material the ditch seemed just as far from completion as when it was begun. The plan was then adopted of running in the trench for one length of pipe, 20 ft., covering that up completely to the natural slope of the slide, and

then excavating for the next section independently, allowing the loose material, with some assistance from the men, to form a rough arch up into the slide above that portion being excavated; in this manner the pipe was successfully laid, a section at a time, down in the more solid portion of the slide, that apparently had not moved for years. It was dangerous work, for in driving the joints the heavy blows of the maul shook the whole slide. The author stood on the bank and watched the movement of the slide while the workmen were down in the ditch, and many times had to give the alarm for the men to jump for their lives. It may be remarked, however, that the whole work of this water system was completed without the least accident to any of the men.

In covering up the pipe through this slide the covering stones were selected and laid so as to form a complete dry arch over the pipe. Last winter entirely obliterated the line of work on top of the slide, but after nearly two years the pipe itself is apparently in perfect condition.

The Pressure Reservoir.—This reservoir was built for two purposes: *First*, as a penstock at the head of the pressure pipe to the mines; *second*, as a reservoir to hold the water for some hours, during the latter part of the season, when the supply was limited. It is not the size or capacity of this reservoir that calls for special attention. Very small results, if they demonstrate principles, may be more instructive than larger enterprises with questionable outcomes.

Plans were made for a larger reservoir, but only the bottom portion was built with a capacity of some 330 000 galls., while if completed to its full height the capacity would be 3 000 000 galls.

This reservoir was formed at the head of a gulch on a thickly timbered part of the ridge above the mines. A semi-circular dam was built across the gulch, of a single thickness of large pine logs, with ends sawed on radial lines and butted close together, forming an arch with a slight batter up stream. Each row of logs was separated with, and bound together by, smaller logs laid at right angles to the dam and extending up into the reservoir. As these were put in place, a bank was formed on the upper side of the timber, of earth and broken rock well tamped in around the logs. This material came from cleaning out the gulch to form the bottom of the reservoir, and was tamped in dry (there was not a drop of water within a half mile) no

attempt being made to form a water-tight or water-holding dam of this material. The slope on the upper side of this dam was one and one-half to one, that on the sides of the reservoir being two to one for convenience, not from necessity. The depth of the bottom portion of the reservoir built was 12.5 ft.

It may be explained here that in order to form the larger reservoir of 3 000 000 galls. capacity, a large amount of material would have to be excavated from the sides. This material, broken rock and earth, it was intended to dump over the present dam, and tamp in layers between the present and a second log dam and crib built farther down the gulch.

The use of these log dams in the manner described was to save expense in handling sufficient material to form an ordinary earth or rock dam, and, as remarked before, would not be adapted to structures intended to be permanent.

In considering the next part of the subject, the most important item of all, the water-tight lining of such a reservoir, the method of constructing the dam bears an important relation to the material used for lining. In this portion of the work, the building of the dam and reservoir proper, it is thought that the plans followed are applicable to dam and reservoir construction for permanent works of almost any size, with proper modifications of details for local conditions.

Certain facts peculiar to the locality under consideration led to the adoption of methods almost if not entirely new. There was no material on this ridge that in any way could be used as puddle for a dam, and the bottom and sides of the reservoir were of earth, rotten granite and shattered slate.

To overcome all these difficulties a lining was formed of asphalt concrete, not what is known in the street paving business as asphalt concrete, composed of asphaltum and sand, for clean, sharp sand would have cost $20 per ton to deliver on the mountain, but a true concrete was used of broken stone and asphaltum. The stone was obtained from a porphyry dyke near by, which had been largely broken up by repeated earthquakes, freezing and thawing. This stone was broken into pieces of 2 ins. and under, all the fine material and dust being left in and sufficient fine material added to form, as nearly as it was possible to do so, the theoretically perfect concrete material, where every space between the pieces is filled with smaller

stones even down to fine dust, so that the surfaces of each particle are brought into actual contact with those next to it, and thus only the smallest amount of asphaltum was necessary to bind the whole together in a solid and water-tight mass.

This concrete was prepared in two ordinary pans or kettles from a street paving outfit. The rock was heated and well mixed in one pan, and the asphaltum paste, composed of four parts of California refined asphaltum and one part of crude petroleum, was boiled in another kettle. This boiling hot asphaltum paste was poured with ladles over the hot rock, and the whole well mixed while over the fire with shovels and hoes, until every particle of stone, sand and dust was thoroughly covered with the paste. The concrete was then taken out into hot iron wheelbarrows and put in place in the usual manner, being thoroughly rammed, rolled and ironed or smoothed down with hot irons. This concrete was put on in one layer 4 ins. in thickness, in strips from 4 to 6 ft. in width, and where the strips were joined, the old edge was well coated over with hot paste. After the whole reservoir was lined, it was painted with hot asphaltum paste mixed in the same proportions, but boiled a much longer time, until when entirely cold it was hard and brittle breaking under the hammer like glass, yet tough, elastic and pliable with the least warmth. This painting was done while the paste was very hot and could be ironed down with hot irons. Its thickness should not exceed $\frac{1}{8}$ in.

The results of this work show :

First.—For nearly two years it has stood during two hot summers and one very cold winter without showing a single crack anywhere in the whole structure.

Second.—In one part of the dam, where a trench was cut through it and then filled up, before the lining was put on, the bank settled some 5 or 6 ins. in a strip about 4 ft. wide, and the lining followed the settling, conforming itself to the new conditions without the least break.

Third.—This concrete, formed and applied as here described on slopes of one and one-half to one, has so far shown no tendency to creep, one of the great objections to the ordinary material formerly used, composed of sand and asphaltum.

Before drawing some deductions from the experience derived from this work, it will be proper to record one or two errors, for failures and mistakes are as instructive as successes.

This reservoir lining was put in by contract, and after a large part was completed, the contractor's "practical foreman" concluded that the fine particles and dust used up too much asphaltum. Hence, without the consent or knowledge, and afterwards against the protest, of the author, he sifted out the finer particles, attempting, in his practical economy, to fill these larger spaces with asphaltum, by painting it over with a heavier coating of paste. It is hardly necessary to say this was an entire failure. The paste when hot ran down into these crevices and left fine holes, which of course leaked. To remedy this defect water was turned in with a large amount of sand and earth. This filled the openings and all leaking stopped. The water was drawn off, the surface allowed to dry, and then a second and third coating of paste was applied. While in this case the result was satisfactory for the time being, such work could not be used in a large storage reservoir, or in a more permanent structure.

On the portion repaired in this manner a much too thick painting was put on, and this outside coat, to which some imperfect sand was added, shows the usual tendency to creep or slide, while in no case, even in the imperfect concrete under it, does this creeping show in the least, but the whole of the concrete, after nearly two years, is in as perfect condition and form as the day it was put on. It is proper to state here that this subject of lining reservoirs with asphaltum was first brought before the Society by J. D. Schuyler, M. Am. Soc. C. E., in his paper* on "The Use of Asphaltum for Reservoir Linings."

The so-called concrete which was used in that work was composed of clean sand and asphaltum, the usual composition for street paving. In the discussion† of that paper some important questions were asked, which, as far as known, have not been answered.

Perhaps the experience here related and the following conclusions may somewhat answer those questions.

First.—It is believed that an almost perfect lining for reservoirs, of any size and in any part of the country and any climate, excepting perhaps under the equator, may be had by using an asphalt concrete such as described in this paper, and at a cost much below ordinary methods of constructing water-tight reservoirs. In this instance a lining 4 ins. thick was put on for 15 cents per square foot.

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* See the *Transactions* of the American Society of Civil Engineers, Vol. XXVII, p. 629.

† See the *Transactions* of the American Society of Civil Engineers, Vol. XXVIII, p. 130.

Second.—The concrete should be formed of broken stone (not gravel or sand) in 2-in. cubes and under, with finer particles down to sand and dust, so that every particle will be in contact as perfectly as possible, thus requiring the use of a minimum amount of asphaltum. If this concrete is made in proper proportions and thoroughly mixed, it can be so compressed by hot ramming, rolling and ironing as to be impervious to water throughout its whole mass.

One element of economy, it is believed from some experiments made, comes from the fact that material for such a concrete, not here referring to the asphaltum, can be had cheaply on any reservoir site where rock is at hand. It is not necessary to go to great expense to procure clean sharp sand as is usually specified for hydraulic cement concrete. The finer particles may be composed of even good sandy loam, if clear of roots and vegetable matter, provided the whole mass is properly dried, pulverized, and used in the right proportions.

Third.—The proportions and preparation of what is here called the "asphalt paste" is a very important item. It is to be regretted that the works built were not of such magnitude and importance as to justify extended and careful experiments on this point, and it is hoped that the author or some other member may have the opportunity of making such experiments and reporting them for the benefit of the society. The time required for cooking this mixture to produce a tough, hard, rubber-like material when almost cold and almost as brittle as glass when absolutely cold, cannot be given in exact figures. Of course great care should be taken not to burn the paste while cooking. It is believed that these two facts, the use of a true concrete instead of a porous sand mixture, and a more thorough cooking of the asphaltum paste, will do away with nearly all of the objections to using asphaltum for reservoir linings, especially the creeping of the lining down the banks, and for this reason steeper banks may be used.

Fourth.—One great advantage thus gained is the possibility of forming dams and embankments for reservoirs of material best suited for the purpose of a gravity dam, and entirely eliminating the question of core, puddle or any water-tight method in their formation.

As a basis of cost of the concrete (with such work as is here described, 4 ins. thick, costing 15 cents per square foot), it is estimated that with asphalt at $20 per ton, and working on a large scale, mixing and handling being done by machinery, such concrete can be put in place

and finished 6 ins. in thickness for the same price, 15 cents per square foot, common labor being at $2 per day.

Suggestion for a New Method of Dam Building.—The question has been asked how far one would be justified in following such a mode of construction in large storage reservoirs. The author will state that he has already recommended the building of a storage reservoir in southern California with a dam 120 ft. high on a plan which to him seems to have many advantages in economy, safety and permanence. The recommendation without details is as follows:

After clearing the dam site to bed-rock in the cañon, which is perhaps a quarter of a mile in width at the top level of the proposed dam, with rocky sides and bottom, and preparing proper toe catches in the bed-rock, a gravity dam is to be built of loose rock blasted from the sides of the cañon and dumped in place by cableways.

Care should be taken in placing the larger rocks, which in this way can be handled up to several tons in weight, so as to surround them with smaller pieces, making the mass as compact as possible and reducing the amount of settling and movement to a minimum. This can be done with but very little extra expense beyond a systematic and intelligent dumping of the rock from the grabs and buckets.

The inner face of this dam should be laid up more carefully. A thickness of perhaps 10 or 15 ft. at the bottom and reduced to from 2 to 5 ft. at the extreme top should be placed by hand in the form of a well laid dry wall, on such a slope as, after investigation of details, would be best adapted to the conditions of the special case in hand. The joints of this hand-laid portion should be well filled with spalls, and the surface, considering the direction from side to side across the face of the dam, left as even as can be done without hammer dressing, though not smooth.

Starting from the bottom, as this hand-placed portion is laid up, every 5 or 6 ft. the surface should be stepped back, say, 3 in., leaving a series of 3-in. steps all the way to the top of the dam. On the surface thus prepared, place a true asphalt concrete perhaps 1 ft. in thickness. If the concrete is properly proportioned, well mixed and well laid, with the addition of the 3-in. steps, there will be no difficulty experienced from creeping.

The advantages of the method here recommended are believed to be:

First.—A perfectly safe and permanent dam can be thus built at a minimum cost at the point proposed. With such a lining a perfect joining can be made to the bed rock in the bottom of the cañon and to the side walls from top to bottom. Such a lining would be perfectly water-tight, but in the case of a leak repairs could be made at the smallest possible cost, not only from the nature of the material, but from the fact that the core would be on the outside and thus easily reached. One other source of economy would be the fact that every particle of the material in the dam would exert its full influence in gravity, none of the rock between the usually placed core and the inner surface of the dam being in submersion.

Second.—Such a structure would be permanent from the fact that all the material composing it would be practically indestructible.

Third.—Safety for very many reasons is a most important consideration. Such a structure would be absolutely safe if the dam were properly proportioned. No material in its construction would be affected by the elements, or by animals boring into or through it. In case of possible leaks by cracks from any cause in the asphalt lining, the water coming into the dam proper would be quickly and completely drained away, a very desirable result.

With even a number of cracks formed in the asphalt lining, there would be little or no danger of a great flood caused by the rapid enlarging of the opening, as has been the case in earth and even in cement laid masonry dams. The passing of all the water in a full reservoir through a crack in such an asphalt lining would have very little effect in enlarging the opening. From some personal and severe tests the author has found that such a concrete, with sand and gravel and water passing over it, is practically indestructible as compared with solid granite. The only way a crack would become enlarged would be by the pressure of the water passing through it, but as the pressure would be against the concrete, backed by the solid rock, this effect would be very small. For these reasons it is believed that, considering the results obtained, such a method of construction would be economical, permanent and safe.

The author is aware that there are many places where rock in such quantities could not be obtained cheaply. With his confidence in such an asphalt concrete, as here described, he would not hesitate to use earth and loose rock in the construction of the dam, and in places

where no rock bottom could be found, with perhaps some additional precautions, in placing the lining. But in the great western arid region where storage reservoirs for irrigation purposes are becoming more important and necessary every year, rock is the cheapest material to be had in almost all instances. Throughout this section, both north and south, and also in many parts of the eastern states, the author believes loose rock gravity dams with asphalt concrete lining to be the best and most economical means of securing efficiency, safety and permanence.

The author would make one more suggestion, drawn from his experience in using this asphalt concrete in lining sluiceways. Asphalt concrete is believed to be the best material for lining earth, gravel or shattered rock ditches, sluices, spillways, etc., when high velocities are necessary, and the banks, sides and bottoms must be protected from the action of the water. Especially is it valuable in repairing overflow dams, spillways, etc., since an absolute union can be made with the original structure, of whatever material it may be composed, a result most difficult to accomplish with the best cement, concrete or mortar.

It is hoped that these suggestions may bring from members of the Society and others their experiences in such work, and a full criticism of the methods herein proposed.

DISCUSSION.

P. W. Henry, Assoc. M. Am. Soc. C. E.—Asphalt blocks such as are used for paving are made with a concrete similar to that described in the paper. The rock is crushed and all the broken stone, ranging in size from 1 in. to an impalpable powder, is used. The results vary according to the kind of rock used; with soft limestone the blocks have not been so satisfactory as with trap and granite, which are now employed exclusively. Mr. Henry.

Experiments have been made with asphalt paving mixtures, using Portland cement and Rosendale cement in place of the ordinary stone dust, but it is uncertain whether there is any advantage in such a practice.

Edward P. North, M. Am. Soc. C. E.—There is no statement in the paper as to the proportion of asphalt or bitumen used in the concrete. The author stated that the finer particles may be composed of good sandy loam, but the speaker believed that whenever there is any clay in combination with asphalt, it tends to produce disintegration. In asphalt pavements it is rather necessary that pure sand should be used with Trinidad bitumen; California bitumen is purer and may stand a larger proportion of clay without injury. He considered that there is ultimate danger in using any clay in combination with bitumen intended to resist weather or wear. Mr. North.

With regard to the use of maltha, it has been found in Europe, where asphalt proper, an amorphous limestone naturally impregnated with bitumen, is employed in contradistinction to bitumen as in this country, that the best results are obtained with *goudron*, which is very like the California maltha. The best *goudron* came from sandstone rocks in the Pyrenees, but this source of supply is now exhausted, and shale oil, or Strasburg grease as it is sometimes called, is used instead. This is a rock bitumen which flows like molasses and was said in Europe to produce a better asphalt than that made with still bottoms. He did not know until recently that the California bitumen with maltha had been used practically and commercially in this country for paving streets.

Boiling the asphalt to drive off the lighter oils, while it will undoubtedly produce a harder product, may not be as good a line of experiment as adding lime dust. Whether the asphalt will deform or not depends upon its temperature; if cold enough, even the softest kinds will hold their position. Experiments made by the speaker some years ago showed him, so far as they went, that the addition of lime dust, or, perhaps better, hydraulic cement, would keep any mixture of asphalt or bitumen from losing its shape more successfully than by driving off the lighter oils.

Mr. Whinery. SAMUEL WHINERY, M. Am. Soc. C. E.—The speaker thought that there might be a danger in rock-fill dams of the loose rock settling away from the asphalt lining when the latter was very brittle, thus breaking the water-tight coating and possibly leading to serious leaks. A large number of tests of asphaltic mixtures made under his direction indicate that had the author mixed with the asphalt from 10 to 15% of very finely pulverized limestone or clay, the strength of the concrete would have been increased and its brittleness greatly decreased. When asphalt pavements were first begun in this country, experiments were made with a number of asphalts, one of the earliest being a nearly pure Cuban bitumen. It was found in practice to make a very brittle pavement, so brittle that it soon disintegrated under the horses' feet. Further trials showed that the Trinidad asphalt is very well suited for that purpose, its superiority being doubtless due to some extent to the fact that it contains a large percentage of very fine silicious and aluminous matter, which is found very intimately mixed with the bitumen. It has been shown by laboratory experiments that Cuban, Bermudez and Californian asphalts in their pure states are exceedingly brittle, but by mixing with them 10 to 15% of powdered limestone, the brittleness and the tendency to flow or creep on account of changes in temperature are decreased.

The preparation of concrete with bituminous cements is not new, and the same principles apply to it as to the making of hydraulic cement concrete. The author is right in saying that it is desirable to have the mass as nearly solid rock as possible. The fragments of rock should be of different sizes so that the smaller will drop into the voids between the larger, reducing the interstitial space as much as possible, and consequently the amount of asphaltum cement required. Concrete of this character has been made for street foundations for a number of years and has been found entirely successful, although for that purpose not so large a proportion of bitumen is used as would be necessary were it desired to make it water-tight. The speaker regarded its expense as a serious matter in reservoir construction. While the cost of this lining, 15 cents a square foot, seems comparatively small, it amounts to over $12 a cubic yard. Unless ordinary puddling were very difficult and expensive to obtain, it is a question whether an equally good and tight dam could not be made with a clay puddling at a less cost. In the case mentioned in the paper it seemed that the material for making a water-tight lining on the inside of the dam was absolutely unobtainable, and doubtless the construction was the best which could be adopted under the circumstances.

With regard to the use of maltha in place of petroleum as a flux for bitumen, there is a popular but erroneous idea held by many people interested in the sale and use of asphalts that California maltha, which is a thin asphalt of about the consistency of tar, is a much better ma-

terial with which to temper and soften bitumen than petroleum. The theory is that since asphalt is composed of two substances, petroline, which is readily made fluid, and asphaltine, which is practically brittle and is not soluble in certain solvents, the maltha acts as a solvent for the asphaltine and converts it into a cement equal in all respects to the petroline. Careful experiments in the laboratory will show this to be entirely incorrect. Asphaltine is as absolutely insoluble in maltha as it is in water, and for that matter it is insoluble in the residue of petroleum stills. Mr. Whinery.

ROBERT CARTWRIGHT, M. Am. Soc. C. E.—The speaker had seen 30-in. riveted pipe in California, which was put together like stove pipe, and, although no rivets were used at the joints, it held together, even when curves were turned. One of these pipes was in service under a head of about 1 000 ft. The sheets were $\frac{1}{16}$ in. thick at the upper end, and gradually increased in thickness to $\frac{3}{8}$ in. at the bottom. In laying such pipes one end of a section is flared a little by hammering so as to form a socket, and the end that is to be inserted in it is drawn a little so as just to enter the socket. The lengths are dipped in a bath of asphaltum at the shop. In jointing the lengths together a swab dipped in petroleum or maltha is wrapped around the socket and then fired. The heat expands the metal, the male end of the next pipe is then inserted in the socket, and the pipe is driven home with a maul or battering ram. The same method is followed in turning curves. The jointing is done so that the longitudinal seam where the plate laps over on the outside section is flush against the seam on the inside section. Such pipes are laid with the male end up stream, and to prevent leakage earth is dumped into the water that passes through the main first. This earth collects in the seams and joints, and is forced into them by the pressure of the water, eventually preventing all leakage. Mr. Cartwright

There is rarely any provision for expansion and contraction in these pipes, as the divergencies from a straight line furnish a means of compensating for such changes in length. The speaker had found in his own practice that the expansion and contraction of wrought-iron pipes 7 ft. in diameter and 200 ft. long was amply provided for if there was an elbow somewhere in the length. At Rochester, for example, he constructed a 7-ft. pipe to carry water under a head of 93½ ft.; in spite of the fact that it is cooled in winter to the temperature of anchor ice, no leakage occurs, owing to an elbow that takes up the expansion and contraction.

H. D. BUSH, M. Am. Soc. C. E.—During the years 1893 and 1894, while the speaker was employed in the construction of the Bull Run pipe line now bringing water into the city of Portland, Ore., four reservoirs, one high service and one low service for each side of the Willamette River, were also under construction, principally in excavation. Three of them were lined with concrete and one with brick laid Mr. Bush.

Mr. Bush. in asphalt, all lining being afterward covered by several coats of asphalt. Reservoirs Nos. 3 and 4, on the west side of the river, were built in a ravine, the hills on one side of this ravine being high and the ground full of springs. Some movement of the ground in Reservoir No. 3 took place during construction, and a large block of concrete was placed in the bottom of the reservoir to act by its weight in holding the bottom of the slope in position.

When the reservoirs had been used but a short time, leaks were discovered in No. 3, due to the cracking of the concrete lining, which allowed the water to escape through the ground around one end of the dam. Tunnels were then driven into the hill on a level with the bottom of the reservoir and drain pipes put in. The following extracts from letters on the subject from Isaac W. Smith, M. Am. Soc. C. E., chief engineer of the Portland Water-Works, and D. D. Clarke, M. Am. Soc. C. E., principal assistant engineer, were presented:

"On the west sides of Reservoirs 3 and 4, the soil back of the slopes was saturated with water from springs, and the crack along *A B* (see Fig. 1) was caused by an inward horizontal movement of the surface slopes, together with the pressure of the berm and parapet wall. I send you a memorandum from Mr. Clarke in relation to the asphalt coating. The asphalt runs when too soft, and cracks with the concrete when hard. Small 'checks' in the concrete are filled with asphalt by the pressure of water, but the asphalt breaks and is forced through when there is a crack through the concrete from settlement or other cause. As the asphalt does not prevent leakage through cracks in the concrete and the concrete always cracks on the slopes and bottom of reservoirs, I think it would be best to place the asphalt under the concrete when the soil is solid and clay for puddling cannot be obtained. "ISAAC W. SMITH."

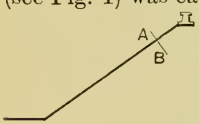


FIG. 1.

"The first coat was of F grade Alcatraz asphalt, put on with brushes or mops while hot. This coat was about $\frac{1}{16}$ in. thick. After being put on, it was smoothed with hot irons, which reduced its thickness somewhat. It was then sprinkled with hot sand.

"The second coat was usually of XXX asphalt, but XXXX grade was used on a portion of one reservoir. This coat was applied hot and was from $\frac{1}{8}$ to $\frac{1}{4}$ in. thick, being somewhat thicker on the bottom than on the slopes, and was not ironed, the thickness of the two coats ranging from $\frac{3}{16}$ to $\frac{5}{16}$ in.

"Some of the reservoirs were not filled at once after the asphalt coating was applied, and with the first warm days that followed the asphalt lining began to creep or wrinkle. This continued until the entire surface exposed to the direct rays of the sun assumed a rough or wavy appearance.

"Later in the season, June 11th and 26th, 1895, tests were made at Reservoirs Nos. 1 and 2 which showed that the asphalt coating would soften and begin to creep when the thermometer indicated 90° at one point and 82° at another. Readings were taken in the sun on the face of the slope.

"On the slopes of all the reservoirs, except No. 1, which was kept filled with water, the asphalt wrinkled badly, numerous places 1 sq. ft.

in area, and over, being almost entirely bare of all asphalt excepting part of the first coat, which had been ironed on. Mr. Bush.

"The work done on Reservoir No. 3 during August and September of this year, 1895, consisted of a single coat of XXX asphalt. On the slopes it was applied about $\frac{1}{16}$ in. thick and $\frac{1}{8}$ in. on the bottom. That used on the slopes was made considerably harder than the coating applied on the bottom. In a few places two coats were applied. So far as observed this work has shown no signs of wrinkling, but the single coat did not prove to be sufficient to protect some small cracks which have since appeared in the concrete lining. These cracks were small, but a second coat of asphalt should have been applied in order to properly cover them. "D. D. CLARKE."

It seemed to the speaker that the asphalt coating as first put on was too thick as well as too soft, while from exposure to the sun during an Oregon winter the asphalt ran down the slopes of the reservoir in wrinkles or folds.

RUDOLPH HERING, M. Am. Soc. C. E.—In the use of asphalt at Babylon and Nineveh, about 4 000 years ago, it was employed as a mortar, and was also mixed with broken bricks and stones to form a concrete, in a manner somewhat similar to that described in the paper. Asphalt was used in Egypt, particularly in Memphis, for keeping moisture out of walls and basements. In a German pamphlet published in the early part of the seventeenth century, mention is made of the application of this material for various industrial purposes, and in 1692 large quantities of bituminous rock were discovered in the Val de Travers, in the canton of Neufchatel, Switzerland. The first practical use made of asphalt in modern times, however, was by a Greek physician, Dr. Eyrinis, who had been requested to make a geological examination of parts of Switzerland and rediscovered the Val de Travers beds. He also found the beds at Lobsann, in Alsace. He experimented with the material, and recommended its use chiefly where its water-tight qualities might be utilized. There are records of cisterns 16 to 20 ft. in diameter, which were laid in asphalt and retained water quite successfully. Count Buffon, the French naturalist, recommended asphalt to be used as a mortar in building a large basin in the Jardin des Plantes in Paris, and some 40 years later he wrote that it had remained perfectly water-tight. Mr. Hering.

The Val de Travers mines were again neglected, as it seemed impossible to accomplish as much with the material as had been expected. In 1802 the Seyssel mines were discovered, and there the rock was richer in bitumen. They were not worked extensively until after 1832, when Count Sassenet took hold of them, and it might be said that the modern practical utilization of asphalt began with his efforts. He found that the natural rock could be improved by mixing with it a natural bitumen or maltha, of which deposits were found in the vicinity. The asphalt mastic made in this way has since then been used extensively for sidewalks in France.

Mr. Hering. An engineer named Merian, located at the Val de Travers, noticed in 1849 that the pieces of rock which fell from the carts into the ruts were compressed and formed a substantial roadway. This is said to have been the origin of the use of rock asphalt for paving, and when the speaker visited the mines a few years ago, this old roadway was shown to him. Mr. Malo developed the Seyssel mines more fully, and, with Mr. Delano, published the best information of the time about the industry, especially with regard to road-making.

In 1843, asphalt mines were discovered at Limmer and Vorwohle, near Hanover, Germany, by a Mr. Henning, who studied the applications of asphalt and wrote, as the speaker believed, the first treatise on the practical use of asphalt for industrial purposes. The German asphalt contained 17% of pure bitumen at that time, while the Val de Travers and Seyssel asphalt contained only 8 to 12 per cent. Mr. Henning recommended employing asphalt to make cisterns water-tight, and advised placing it as a filling between two masonry walls. He stated that it is an excellent material with which to cover inclined surfaces and protect them from the penetration of water, and, if but one coating is put on, he recommended using some textile fabric of hemp or linen as a foundation to keep the asphalt from crawling, omitting the fabric if two coats are employed. He stated further that sand should always be rubbed on the asphalt surfaces, not only to increase the stiffness of the material, but also to give it a lighter color, which reduces the absorption of heat and consequently the flowing or crawling of the asphalt. He even recommended a coat of whitewash to prevent this creeping.

Mr. Henning also stated that the best asphalt will lose its oily substance in time, and recommended that the surfaces should be painted with hot maltha every eight to ten years. He wrote that vertical walls can be coated with asphalt to protect them against moisture, and said that this should be used in the form of pure mastic or bitumen, without any sand. For application to reservoirs, he preferred two thin coats of maltha to a thicker one.

Subsequently there came into general notice the Trinidad asphalt, which has a comparatively high percentage of bitumen, then the Bermudez asphalt, and finally the Alcatraz asphalt. The Bermudez asphalt is said to be richer and to retain its volatile oils longer than that from Trinidad. The Alcatraz mines are located near Santa Barbara, Cal. From one of them, Las Conchas, a very high grade of bitumen mixed with fine sea sand is obtained, while at another mine, La Patera, a rock asphalt is found. For practical purposes in California, a mixture of the two has generally been used.

In a paper* presented to the Institution of Civil Engineers, Mr.

*"The Use of Asphalt in Irrigation Works in California," *Proceedings* of the Institution of Civil Engineers, Vol. CX, p. 286.

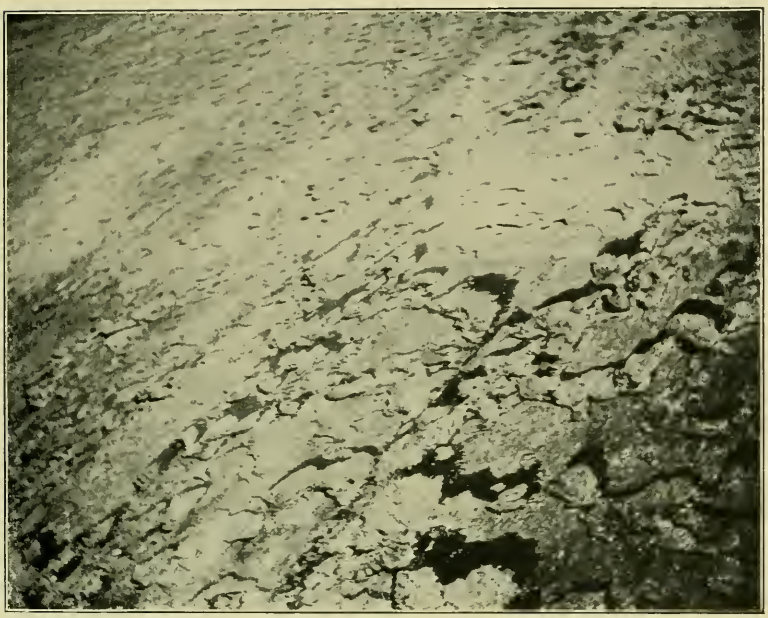


FIG. 1.

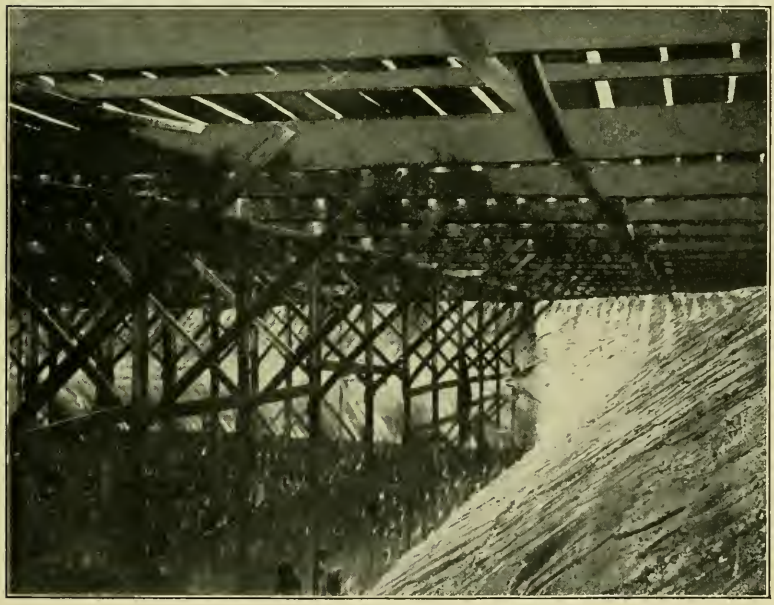


FIG. 2.

Gervaise Purcell recommended a reservoir lining of three parts of gravel and one part of sand, to which 10% of asphalt mastic has been added. He heated the gravel and sand in a horizontal cylinder to a temperature of 310° Fahr., added the asphalt at a slightly lower temperature, and then turned the whole mass over so that the materials were thoroughly mixed. The mixture was tamped against the slope and not rolled, because the rolling was said not to compress the material sufficiently to remove all the interstices. The speaker preferred the use of crushed rock to gravel, as the concrete prepared with it would be less likely to move down the slope. Mr. Hering.

At Portland, Ore., he had examined three reservoirs having thin coats of asphalt on their slopes, and one reservoir on which the slopes were paved with two courses of brick dipped in asphalt. The asphalt coatings had slipped and crawled on the slopes, as shown in Plate I, Fig. 1, but in no case had they entirely left the surface of Portland cement concrete upon which they had been placed. The asphalt was applied in two coats, and it was intended to make the second a little harder than the first; but on account of bad weather this had not been done, and the crawling is attributed principally to this fact. In order to make the asphalt adhere to the concrete, it was found necessary to apply the first coat very soft, mixed in the proportion of one part of asphalt to two and a half or three parts of gasolene, and recent experiments at Philadelphia confirmed this practice. The speaker exhibited a specimen of asphalt lining which had adhered so strongly to the concrete that it had pulled off the cement surface when removed.

The asphalt lining of the Denver reservoirs* was stated to have been unsuccessful. The material was softer than when first laid, and had shrunk and cracked seriously where not exposed to the water. Much of it had crawled down the slope, as shown in Plate I, Fig. 2, even when under water. It was said that if there was any difference at all in the condition of the linings, that made with Alcatraz asphalt was superior to that made of Trinidad asphalt. The speaker's observations and inquiries led him to believe that the best method of applying asphalt for reservoir linings was either to have upon a concrete surface the thinnest practicable coat of asphalt, free from sand, or to use an asphalt concrete, as described in the paper.

John C. Trautwine, Jr., Assoc. Am. Soc. C. E.—Asphalt was used in making repairs to the Queen Lane Reservoir of the Philadelphia Water-Works, which leaked so badly when partly full that it had never been filled entirely. The reservoir consists of a north and a south basin, partly in excavation and partly in embankment, the bottom of the basins being at an elevation of 208.33 ft. above the city datum, and the intended elevation of the water surface 238.33 ft. The bottom and slopes of the reservoir are lined with 2 ft. of somewhat micaceous Mr. Trautwine.

* See the *Transactions* of the American Society of Civil Engineers, Vol. XXVII, p. 629.

Mr. Trautwine. clay, obtained in the neighborhood, which was well rammed, and ought to have held water when the reservoir was filled to the moderate depth of 10 ft., to which it was subjected before the repairs were begun. The clay on the sides is covered with concrete slabs 10 ft. wide, 12 ins. thick at the bottom, 6 ins. thick at the top, and running from the top to the foot of the slope. The first slabs were laid with 10-ft. spaces, which were afterward filled with similar slabs, and it seemed to the speaker that the cement used for the surface finish must have varied in quality, as the slopes appear under certain atmospheric conditions to have alternate light and dark bands. The clay on the bottom is overlaid by a concrete floor 4 ins. thick, the edges of which overlap the foot of the slope.

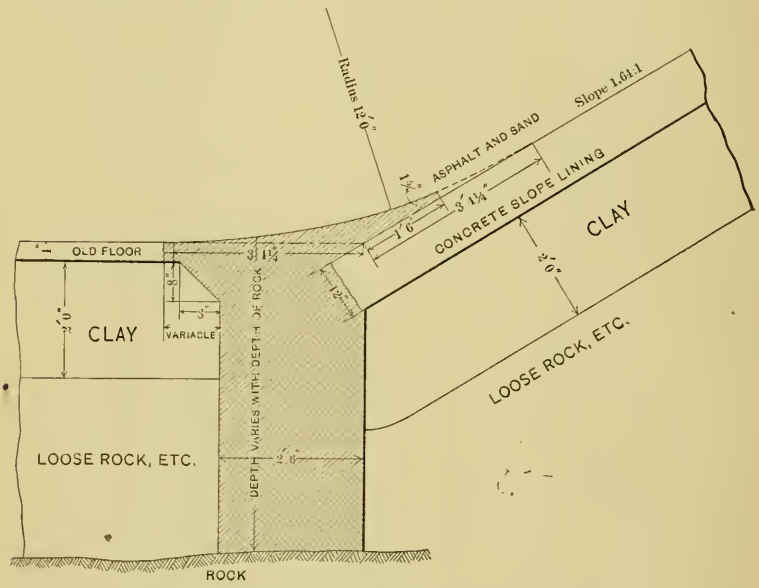


FIG. 2.

The concrete slabs on the slopes cracked across in nearly continuous lines at heights of about 5 and 10 ft. above the bottom, except on a portion of the slope which formed a reversed curve. Before the repairs were made, the rain that fell on the surface of this concrete was fed by the horizontal cracks into the longitudinal joints between the slabs, and passed through these to the clay below. The clay was softened and washed away to some extent, especially near the foot of the slopes. Many of the slabs which were thus left unsupported at the lower portion of their length yielded somewhat under the weight of 10 ft. of water over them, and their lower edges were pressed down so as to leave a space between them and the concrete floor, amounting

in some cases to 1 in. Through this space the water was seen escaping freely, even when its depth was such as barely to cover the space. In some cases the rain water passed down the slope between the concrete and clay, carrying portions of the latter with it, and came up between the slopes and the floor, leaving small mounds of clay within the reservoir. Mr. Trautwine.

The repairs were made by constructing a concrete wall under the foot of the slope of one basin, and by closing the horizontal cracks and seams and the joints of the concrete on the slopes with melted asphalt, which was also used in closing the joints and larger cracks in the floor.

The design of the footing wall is shown in Fig. 2. It is 2 ft. 6 ins. wide, and runs down to a firm foundation on the underlying stratum of micaceous rock, which becomes more or less decomposed as it nears the surface. The wall is about 3 600 ft. long, and extends entirely around the basin. The floor was originally almost level, its inclination being just sufficient to insure drainage. In the new construction, it was finished with a curve tangent to the floor and the slopes, the curved surface being finished with a 1-in. coat composed of equal parts of sand and cement. The small triangle above the concrete was filled with a mixture of sand and asphalt. In building the wall every effort was made to afford a support to the foot of the slope, and this work was made easier by heavy rains which fell while the trench was open. These undercut the concrete slabs to some extent, and it was necessary to ram the concrete with an improvized wooden tool, shaped somewhat like an adze and struck with a sledge while held against the concrete placed under the toe of the slope.

The asphalt repairs were made on both basins. The material used was composed of four parts of Bermudez asphalt and one part of the liquid or F grade of Alcatraz asphalt. These were melted together and poured on the surface to be treated, which had been previously primed with a thin coating consisting of three parts of the F grade of Alcatraz asphalt and seven parts of gasolene. The asphalt was only partly liquid, and it was necessary to melt it before it could be dissolved in the gasolene.

The priming coat was allowed to dry thoroughly before the asphalt was applied over it, in order that all the gasolene might escape. The use of this priming coat was found to be of the utmost importance. Asphalt placed on it adheres quite tenaciously, while if applied directly to the original surface of the cement concrete, it breaks off readily.

The horizontal cracks in the concrete slabs on the slopes and the seams between the slabs were given a single coat of the melted asphalt. At the outset, the seams were given two coats of asphalt with a strip of burlap between them, but this plan was abandoned on account of

Mr. Trautwine. the time required to carry it out. It was also found that the burlap, not being anchored at the top, had a tendency to creep down the slopes and to buckle near the foot. In the basin where the footing wall was not used, the melted asphalt was applied over a width of about 1 ft. on the slope and on the floor, extending each way from the joint between them, and special pains were taken to secure a considerable thickness of the asphalt at the joint itself. In the second basin, the entire surface of the new concrete footing wall and of the triangle of asphalt and sand about it was covered with the melted asphalt, which was made to cover all the joints of the new surface. Two or three large cracks in the concrete floor of each basin were filled carefully with asphalt mixture. As it was necessary to have the cracks and seams in the slopes closed before winter set in, the work was hurried and carried on at times in weather which it was feared might prove later to have been too damp. Two methods were accordingly adopted to dry the concrete locally. The first was the use of gasolene hand torches, and the second was the use of gasoline stoves, such as are used for softening asphalt pavements in repairing streets. This type of stove consists of a sheet iron shield or reflector placed a few inches above the concrete and having below it a number of gasolene jets or burners.

Before the repairs were begun, the reservoir lost water from all causes at the rate of about 0.4 in. a day from one basin, and 0.9 in. from the other, under a head of 5 ft., while since the repairs, the total loss under the same head amounts to from 0.1 to 0.25 in.

CORRESPONDENCE.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E.—Asphalt lining for new reservoirs and for repairing old leaky ones has been in use for many years in California and adjoining States. The results obtained so far are most encouraging. For the bottom and side-slopes flatter than 1½ to 1, the best mixture is either asphalt mortar or asphalt concrete. It is the cheapest and best lining, and there is no danger of it crawling down the slopes. For steeper slopes up to vertical faces, this kind of lining has been tried and found wanting in many respects. Under a hot summer sun it will creep down the faces in spite of all precautions.

Steep slopes or vertical walls are now coated as follows: *First*, with a cold liquid asphalt paint, which has great penetrating and adhesive properties, but is lacking in sun-proof qualities; *second*, with a heavy layer of ordinary burlap, which is tightly stretched and pressed into this liquid asphalt paint; *third*, with a heavy outside coat of hard asphalt

paint, put on boiling hot. This constitutes the weather coat, and is hard, tough and resists the hot summer sun admirably. Wherever this lining has been used, no signs of creeping have developed, even on smooth vertical faces. Hard asphalt paint is lacking in adhesive qualities, and consequently cannot be placed directly on the slopes. The contract price for this lining has varied from 12 to 16 cents per square foot, depending upon local conditions. Mr. Le Conte.

Dry-stone dams have been in use in California for storage purposes, in mining districts, during the past 30 to 35 years. A sketch of the characteristic section is shown in Fig. 3. These dams require about three times the cubical quantity of stone which would be called for in a properly designed masonry dam, but the first cost, $2 to $3 a cubic yard in place, is so low that in many places they have performed a

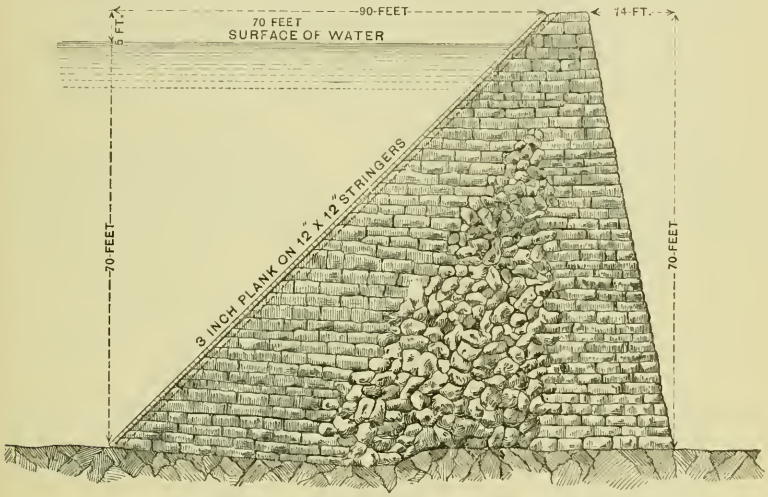


Fig. 3.

most useful purpose. These dams are lined on the water side with 3-in. plank, sometimes with two thicknesses. The foundation in all cases is solid rock free from fissures.

While it is proper to give these dams full credit for all past and present usefulness, yet it is hardly right to call them perfectly safe and permanent. Past experience shows clearly that whenever a serious leak develops, or should the dam be overtopped by flood waters, the dry-stone structure will be literally blown away, like chaff before the wind. After the failure of the English dam on Yuba River, in California, in June, 1883, not a vestige of stone was found for a distance of 1 500 ft. below the site of the dam, and all large stones used in construction, each weighing several tons, had disappeared entirely; probably they were ground into smaller fragments.*

* See the *Transactions* of the Technical Society of the Pacific Coast, Vol. II, p. 3.

Mr. Le Conte. The author's suggestion of substituting asphalt concrete in place of the wooden planking as a water-tight lining for the up-stream face of the dam seems to be reasonable, but experience alone can determine whether it is better construction. This type of dam is eminently suitable for large storage reservoirs in remote districts where civilization is not likely to encroach upon the valley below the site. They are cheap, practical and within certain limits reasonably safe, but like earth dams they require constant supervision and quick attention.

Mr. Davis. A. P. Davis, Assoc. M. Am. Soc. C. E.—In 1894 the writer was associated in the formulation of plans for a large irrigation system in California, in which canal velocities of from 8 to 12 ft. per second were required, and it was decided to line the canals with concrete, composed of sand, broken stone, and sufficient asphalt of high grade to form a good bond. It is gratifying to find that the author's experience so emphatically vindicates that decision. The side slopes of the canal were to be 1 to 1. In the same project were included the plans for a rock-fill dam 120 ft. in height. The construction here recommended was similar to that described in the paper, except that the water-tight face of the dam was to be a double floor of redwood, fastened to timbers built in the rock and properly calked. An asphalt concrete facing was carefully considered, but reluctantly abandoned, for the following reasons:

In the absence of satisfactory evidence to the contrary, it was thought that a sheet of asphalt concrete of such magnitude would be sure to creep sufficiently to impair its efficiency seriously, unless the slope were made so much flatter than demanded for a timber face as to enhance greatly the cost of the rockwork, and, even then, the cost of repairs, owing to the difficulty of access to the site of the dam, would be so great as to render new experiments rather hazardous. It is to be regretted that the author omits to state the slope on which he expected his "outside core" to stand without creeping, and on what experiments he based his expectation. It would seem that his recommendation of 1 ft. thickness is excessive, and tends to increase the danger of creeping, by removing a part of the load a greater distance from its bond with the rough rock face.

This is an important question in the arid southwest, and further light on the subject is badly needed, especially as to the greatest slope at which it is possible to place this concrete safely without impairing its adaptability to the settlement of its backing. As to the exact composition of the concrete, and the time and method of cooking, it would seem to be necessary to make experiments for each individual structure, and even for each consignment of asphalt, as the composition and physical properties of even the best asphalts vary somewhat.

Mr. Cummings.

Robert A. Cummings, Assoc. M. Am. Soc. C. E.—In preparing the design for the floor of a large dry dock, where a strong and impervious but inexpensive construction was required, the writer contemplated using an asphaltic concrete floor as shown in Fig. 4. No data was at hand concerning the strength of such a concrete to resist the hydrostatic pressure, or how it would act as a monolith under temperature stress, or what its length of service would be when alternately wet and dry. It was assumed that the flexibility of an asphaltic concrete was greater than that of cement concrete, and that this would render it less liable to crack where it was attached to the piles and timber framing. It also appeared to have other recommendations suggesting its suitability for a dry-dock floor.

The design, however, was modified as shown in Fig. 5. In this case relief from hydrostatic pressure is partly obtained by drainage through the rip-rap and broken stone foundation of the floor.

The tops of the piles and timber framing are flush with the top of the broken stone. Over the top of the broken stone, piles, and timber

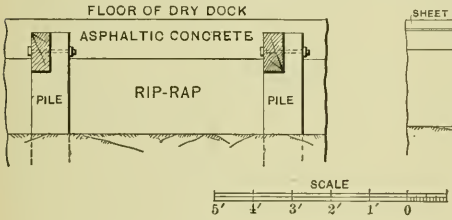


Fig. 4.

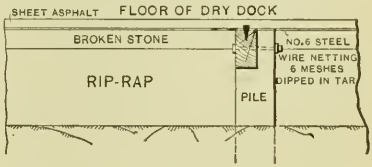


Fig. 5.

framing, is laid a No. 6 steel wire netting with 6-in. meshes. This netting has been previously dipped in boiled coal tar. It is then securely fastened by staples to the piles and timber framing. The netting is introduced to add strength and to act as a binder to the asphalt finishing course above.

The specifications of the asphalt finishing course are the same as are used for ordinary street paving. The writer believes that this floor admits of being modified to suit the lining of reservoirs. He desires to ask what is the effect of sea water and brackish water upon asphaltic compositions. He also inquires what is the condition of asphalt linings at the water line in reservoirs after two or three years' service, also what is their durability.

Mr. Stanton.

Robert Brewster Stanton, M. Am. Soc. C. E.—Before considering some of the points brought out in the discussion of his paper, the author desires to state that it was not his intention to claim as original or even new either a rock-fill dam or an asphalt lining for reservoirs; but if there was anything really new in his suggestions it was the combination of the several items for storage reservoir dams, and especi-

Mr. Stanton. ally the use of a true asphalt concrete, which it was believed had never before been used for such purposes. It is certainly of advantage to have drawn out some information from the experience of other engineers and such historical facts as given by Mr. Hering. It is to be regretted, however, that others have not given facts and data in their possession for the benefit of the Society.

A number of members and others during the past few years have made extensive and valuable experiments in the use of asphalt (especially in the amount and proportion necessary for a perfect mixture) and also in the construction of modern rock-fill dams. The author personally requested several of them to give their data and experience in the discussion of this paper, and regrets exceedingly they have not been able to do so.

Mr. Hering's historical facts are both interesting and instructive, It is very gratifying to learn that the large basin in the Jardin des Plantes in Paris, built with asphalt, "had remained perfectly water-tight for 40 years," and, if the opinion quoted from Mr. Henning be correct, that the best asphalt will lose its oily substance in time, and should be painted with hot maltha every 8 or 10 years, it is a strong argument for the outside core suggested in the paper.

One of the most remarkable uses of asphalt for the core of a dam is in the work described by Mr. Skinner, in the discussion of Mr. Van Buren's paper on "High Masonry Dams."* The author very recently visited and carefully examined the dam there referred to, then being built in Southern California. It is not the intention to discuss that work, except so far as the asphalt used forms the entire effective core of the dam, and shows most questionable engineering practice. As described by Mr. Skinner, this dam is a loose rock-fill dam, and when completed will be 130 ft. in height. In the center of this is placed a core "of thin riveted steel plates from No. 0 to No. 3 Birmingham gauge. As each course of plates is calked and completed it is coated with a special preparation of asphalt (pure, soft Alcatraz asphalt). Burlap is then placed over the top edge and on each side of the steel, and a coat of (pure, hard Alcatraz) asphalt put over it, making two coats on each side of the plates. On each side of the web built up in this manner is a 24-in. (to a 12-in.) hydraulic cement concrete wall," with the rock-fill dam built against it. This is called a steel core, but looking at this dam as a permanent structure (and it certainly should be permanent, built as it is at the head of a large agricultural valley, and for the purpose of supplying water to a future large city) it is, more properly speaking, an asphalt core, for the following reasons :

The concrete walls on either side of the steel can in no way be depended upon as part of a water-tight core, subject as they are to the strains from the settling of such a rock-fill dam, and being in an earth-

* See the *Transactions* of the American Society Civil Engineers, Vol. XXXIV, p. 506.

quake country. This concrete is placed in the manner and position where it is, simply to protect the asphalt from injury by the filling in and settling of the rock against it. The steel itself, with the water in contact with it (without the asphalt), would have but a limited life, especially in consideration of the nature of the soil and water in Southern California, from 7 to 10 years at the most. Hence, as a permanent structure, the water-tight core of this dam consists of a sheet of asphalt and burlap, say ¼ in. in thickness, standing vertically in the center of a rock-fill dam, this sheet of asphalt being supported in place by steel plates. Mr. Stanton.

The engineering defects of constructing an asphalt core in this manner are:

First.—The entire want of certainty in the permanence (beyond a few years) of such a thin sheet of asphalt in a vertical position in a loose rock-fill dam.

Second.—The placing of such a core, both steel and asphalt, where, financially speaking, it is beyond the possibility of being repaired.

Third.—The absurdly excessive first cost of such a core, which cost, to the present time, has been more than five times what it would have been to put on an asphalt concrete face, such as suggested in the paper. It is gratifying to state that the engineer in charge of this work is not responsible for the defects of plan or the great cost of the work. This belongs to the president of the company, but in the execution of the details, the engineer has shown marked ability and a conscientious painstaking which cannot be too highly praised.

There are three points in the discussion that seem to require notice.

First.—The nature and proportions of the concrete used. The author regrets, as stated in the paper, that he cannot give more detailed data. From some statements made by members of the Society in this discussion, he is more than convinced that the profession is in need of careful experiments and well kept data upon this subject. The following quotations illustrate this point.

Mr. North "believed that whenever there is any clay in combination with asphalt it tends to produce disintegration."

Mr. Whinery says "that had the author mixed with the asphalt 10 to 15% of pulverized limestone or clay, the strength of the concrete would have been increased."

Mr. North says "that the addition of lime dust or, perhaps better, hydraulic cement would keep any mixture of asphalt or bitumen from losing its shape," etc.

Mr. Henry says that from experiments made, "it is uncertain whether there is any advantage in such a practice," that is, in adding cement.

The author does not presume to decide between such authorities, but does express the hope that much more definite and reliable data

Mr. Stanton. from actual work or experiments will be given the Society, and as a contribution toward this he submits his work with a test of two and a half years.

Second.—Permanence. Mr. Le Conte thinks the author claims too much for the permanence of the methods suggested. What he says as to the power of water in washing away great bowlders of several tons' weight is of course correct.

The possibility of such a contingency as the stream overflowing a rock-fill dam is not to be considered. Means to prevent this positively should be one of the first provisions in the plan. The construction of an earth dam like that across the Pecos River above Eddy, New Mexico, which the author visited last summer, where a lake is formed covering 8 331 acres, and containing 45 000 000 000 galls. of water, would be worse than a mistake, it would be a crime, except for the waste ways provided above the dam, with an aggregate cross-section area ten times that of the dam and 10 ft. below its crest.

The following facts may throw some light upon the permanence of asphalt concrete. In connecting the bed-rock cut with the sluice of the hydraulic mine, described in the paper, the space *A* (Fig. 6),

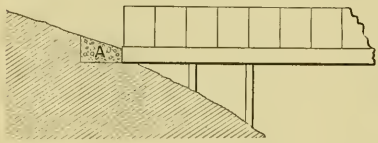


Fig. 6.

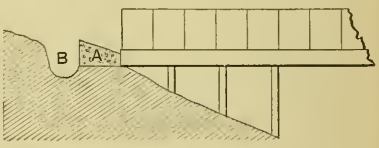


Fig. 7.

between the bed-rock and the timbers of the sluice, was filled with asphalt concrete. Over this in a short time was run thousand of tons of water mixed with sand, gravel and sharp broken rock during the process of hydraulicking.

This work had no perceptible effect upon the asphalt concrete, while the iron in the bottom of the sluice was cut away, and the solid granite above the concrete was worn down to a depth of 3 ft., the full width of the cut as shown at *B* in Fig. 7. Hence it is claimed that through a crack or a hole as large as the wrist, the amount of water in any reservoir, even with sand or gravel, might pass without any enlargement of the crack or hole in the asphalt, or any danger to the dam itself.

The thickness of 1 ft. of concrete was suggested to provide ample material to fill up the unevenness of the rock slope, and the 3-in. offsets. From 4 to 6 ins. of asphalt concrete is entirely sufficient as a water-tight material, but it is believed that a thicker coating would cost less than the amount necessary to form a smooth surface of rock or cement concrete to place the asphalt upon, and a broken stone asphalt concrete properly formed and applied will not creep.

Third.—Definite figures have been asked for as to cost of rock-fill dams. The prices given by Mr. Le Conte of $2 to $3 per cubic yard must have been of those dams built 30 to 35 years ago to which he refers. The following figures are of a recently constructed dam in Southern California where modern machinery and powder were used to do the work, and are correct and reliable. Mr. Stanton.

Data.—Common labor cost $1 75 per day. Coal cost $10 per ton. The plant consisted of one Lidgerwood cableway with three derricks on the dam for distributing and placing rock. Quarrying was done by exploding several tons of powder in drifts and shafts, thus breaking up from 25 000 to 30 000 cu. yds. of rock by one shot. The total amount of rock in the dam was about 120 000 cu. yds. The first cost of the plant was $12 000 in round figures. The cost of the rock in place was as follows :

Quarrying rock.....	6	cents	per cubic yard.
Loading buckets (by hand), including breaking larger rock with powder.....	20	"	"
Hoisting and conveying rock.....	6	"	"
Placing rock on dam.....	3	"	"
Cost of plant.....	10	"	"
Total.....	45	"	"

This includes all repairs, and the total destruction of the whole plant in this one piece of work.

There are local conditions at this particular point where, if proper forethought had been used, this price could have been reduced at least 5 cents per cubic yard, and whatever value there may be at the conclusion of the work in the cableway, the boilers, engines, etc., would still further reduce the cost per cubic yard. The engineer who has so successfully carried on this work, under some very trying conditions, deserves the greatest praise for the skill, thoroughness and economy with which he has executed it.

The author is aware that there are places where rock-fill dams might be advisable, where these figures would be far too low, and yet he is also acquainted with valuable reservoir sites where rock-fill dams can be built (not including preparation of foundation) for 35 cents per cubic yard, with the asphalt concrete face or outer core not to exceed 20 cents per square foot. Even with the extra large cross-section necessary it would appear that in the section of the great arid West especially referred to in the paper, if not elsewhere, this mode of building storage reservoirs would be both economical and permanent, when properly planned and executed.

Vol. XXXV. JULY, 1896.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications

No. 773.

COST OF SEWER CONSTRUCTION, DENVER, COLO.

W. W. FOLLETT, M. Am. Soc. C. E.

PRESENTED JANUARY 8TH, 1896.

WITH DISCUSSION.

The city of Denver, Colo., containing with its suburbs some 130 000 or more people, is built on both banks of the South Platte River, the course of which is here approximately northeast. The main portion of the city is on the south side of the river. The sewerage system has grown with the town, all sewage being discharged into the channel of the South Platte River. Reference to the map (Fig. 1) will show that prior to the construction of the intercepting sewer below Twenty-sixth Street house sewage from the south was discharged into the river at Thirty-sixth Street by the Downing Avenue sewer, at Thirty-first Street by the Thirty-first Street sewer, and at Thirty-fourth Street by a wooden box from Twenty-sixth Street, not shown on the map. Both Downing Avenue and Thirty-first Street delivered small quantities of sewage, the main body coming from the Delgany Street sewer, the end of which was at Twenty-sixth Street. Here some 30 second-feet of sewage, plus a large amount of storm water during short periods of time, had to be taken care of.

During the early spring of 1894 Mr. E. P. Martin, Chief Engineer of the Board of Public Works, designed an extension of the Delgany Street sewer (see Fig. 1) to serve as a main intercepting sewer, carrying all sewage from the south down the river to the outskirts of the city, and to be built on such a gradient as to eventually reach the surface of the ground, some 3 miles below Twenty-sixth Street, at which point the sewage would become available for irrigation and be kept from the channel of the river. This ultimate disposal of the sewage is rendered necessary from the fact that practically the whole summer flow of the river is taken out into irrigation ditches above Denver, and the channel in September is nearly dry. Sewage collects on sand bars and becomes a menace to health.

In May, 1894, the Board of Public Works, consisting of Arthur C. Harris, president, T. B. Buchanan and C. W. Rhodes, determined to build some 8 300 ft. of the Delgany sewer extension—as much as the funds at their disposal justified—and they decided to do the work by day labor, instead of by contract.

It is the purpose of the author to present statements of actual cost of completed work, together with such data as to wages paid, cost of materials used, etc., as will enable members to compare the cost of this work by day labor with contract prices.

The drainage areas and the estimated amounts of house sewage and storm water from them are as follows:

	Acres.	Second-feet.
House Sewage.		
Above Twenty-sixth Street in main	6 200	77.8
Entering at Twenty-sixth Street	320	3.7
Entering at Thirty-first Street	1 545	18.0
Entering at Thirty-sixth Street	2 520	29.4
Total	10 585	128.9
Storm Water.		
Above Twenty-sixth Street	255	159.6
Entering at Twenty-sixth Street	45	28.8
Total	300	188.4

Storm water from a portion of the business section of the city is carried to Thirty-first Street, where a spillway was built to empty a

portion of the flow from above into the river. Of course water flowing over the weir of the spillway will be diluted sewage, but this flow occurs rarely. Twice only during the summer of 1895 did water go over the spillway, and then only for a few hours each time. The construction of the spillway is shown in Fig. 2.

Manholes were built about 400 ft. apart along the whole length of the extension. In the first one below the spillway, 400 ft. down the sewer, a gate was built, which can be lowered at any time and so force over the spillway all the sewage except that entering at Thirty-sixth Street. This gate was in use from January 24th to June 7th, 1895. Its cost, as well as that of the special manhole in which it was placed, is included in the cost per foot of Section 3.

After considerable delay, owing to difficulties of a legal nature which had to be overcome, work was commenced about the middle of August, 1894, under the direct charge of Mr. A. M. Gibson, who perfected an admirable organization of the forces, and remained in charge of the work until December 7th, 1894, when he resigned to go into other business, and was succeeded by the author, who remained in charge until the work was completed in June, 1895.

Construction was carried on through the winter when the weather was not too inclement, thus adding materially to the cost of some portions of the work, principally that of the spillway and of Section 3, but helping out the laboring men of the city when help was badly needed.

The work was divided into sections, the division points being so located that the kind of work on each section was uniform over its whole length. A careful record was kept of all material coming on to the work, showing on which section and for what particular class of work on the section it was used. The foremen made daily reports, showing the disposal of their forces. At the end of each month these daily reports and distribution accounts were checked against the payrolls and bills for material and made to agree with them. A monthly distribution sheet was made up for each section on which work had been done during the month, each section being assigned its proper share of the general expenses (the cost of the engineering force, bookkeeper and watchmen), and as each section was finished, these monthly statements were consolidated into a final statement.

When the work was completed there were fourteen of these final statement sheets which together covered the total amount charged on

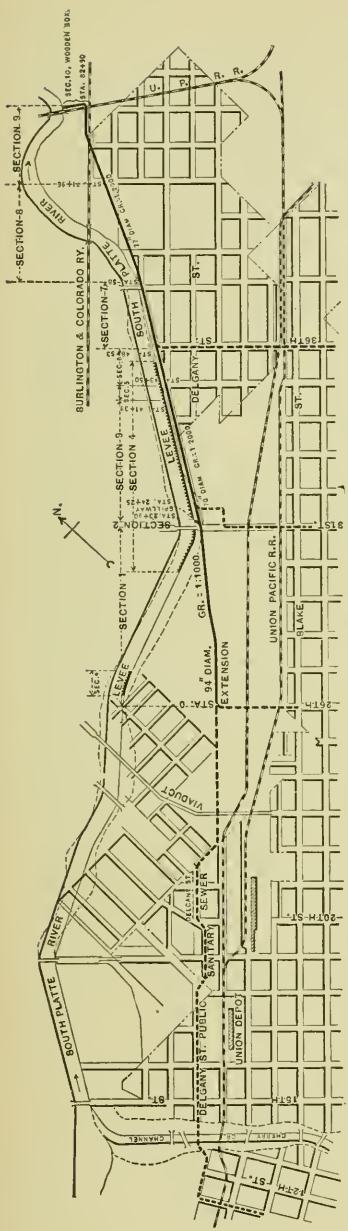

SECTION 9
SECTION 8
SECTION 7
SECTION 9
SECTION 4
SECTION 2
SECTION 1
SURLINGTON & COLORADO RY.
SOUTH PLATTE RIVER
U. P. R. R.
DELGANY ST.
LEVEE
GR. = 1:1000
94" DIAM. EXTENSION
UNION PACIFIC R.R.
BLAKE
VIADUCT
SANITARY SEWER
DELGANY ST. PUBLIC
UNION DEPOT
CHANNEL
36TH
31ST
26TH
20TH ST.
16TH
12TH ST.

Fig. 1.

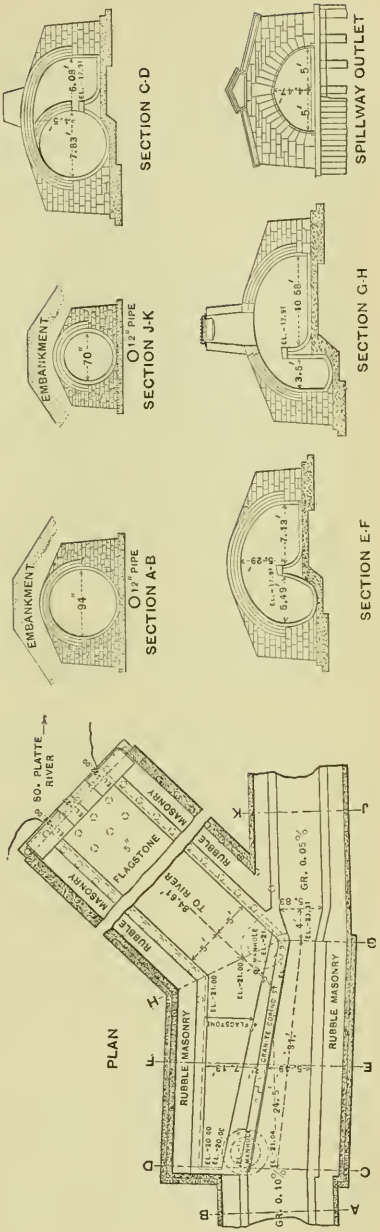

SECTION C-D
SPILLWAY OUTLET
SECTION J-K
SECTION G-H
SECTION A-B
SECTION E-F
EMBANKMENT
12" PIPE
PLAN
SO. PLATTE RIVER
FLAGSTONE
RUBBLE MASONRY
TO RIVER
GR. 0.05%
GR. 0.10%

Fig. 2.

the city auditor's books to "Delgany Street Public Sewer Extension," up to July 15th, 1895. After that date a small amount may have been added to the account to cover the cost of completing the record maps, etc. The following table shows the amount carried on each sheet, and reference to the map (Fig. 1) will show the sections covered by each sheet, except the minor ones for connections, etc.

Sheet.	Section.	Sewer and Description.	Length.	Cost per foot.	Total cost.
1..	.....	Charges against sewer not chargeable to per foot cost	.....	.....	$16 911 93
2..	.....	Extra work on Section 1 outside of net section	.....	.....	2 131 41
3..	1	94 ins. diameter, stone cradle	2 393.7	$20.191	48 331 71
4..	2	Spillway	35	.....	6 774 70
5..	3	70 ins. diameter, stone cradle	1 714	16.515	28 306 95
6..	4	River protection work	.....	.....	9 687 90
7..	5	70 ins. diameter, in rock	211	9.41	1 985 69
8..	6	70 ins. diameter, three-ring work, in rock	503	10.816	5 440 61
9..	.....	Downing Avenue sewer connection, Thirty-sixth Street	.....	.....	505 42
10..	7	77 ins. diameter, in rock, three-ring work	947	12.567	11 900 30
11..	8	" gravel, "	1 396	12.292	17 158 65
12..	9	" sand, "	1 094	12.956	14 173 59
13..	.....	Consolidation of sheets 10, 11 and 12, average	.....	12.58	.....
14..	10	Wooden box, 328 ft. long	.....	6.91	2 231 25
		Total amount charged to the extension			$165 540 11
		Deduct sheet 1			16 911 93
		Cost of construction work			$148 628 18
		Deduct sheet 14, temporary wooden box			2 231 25
		Total cost of 8 293.7 ft. of permanent work, including cost of spillway and protection from river			$146 396 93

The items on Sheet 1, aggregating $16 911 93, while part of the cost to the city of the improvement, are not properly chargeable to the per foot cost of the sewer, being legal charges, cost of preliminary surveys and of work on record maps, expenditures for right of way, etc. Deducting this amount from the total charge of $165 540 11 leaves $148 628 18 as the total cost of construction work, including $2 231 25 spent on a temporary outlet. Deducting this leaves $146 396 93 as the cost of 8 293.7 ft. of permanent sewer, including $9 687 90 spent on a levee some 3 200 ft. long, heavily slagged, to keep the river away from the sewer.

The following brief description of each section is given as explanatory of the conditions under which the work was done.

Section 1, 94 Ins. Diameter, 2 393.7 Ft. Long.—This section was built on ground the original elevation of which averaged a little above invert grade, but which had been filled in with refuse to a depth of from 2 ft. to 5 ft. This filling consisted principally of manure, but was full of tin

cans, wire, old bed springs and the trash which accumulates on a city dump, making the cost of excavation high. The bottom of the trench was from 2 ft. to 4 ft. below the water level in the river close at hand, and the material was coarse gravel, so there was a large amount of water to pump. The back-filling was expensive, as a large proportion of the material had to be hauled in on wagons. While Table No. 1 shows the back-filling less than the excavation, the material taken from the trench was not suitable for covering the sewer.

The section had a concrete base 16 ft. wide and 8 ins. thick, and a stone cradle containing 1.8 cu. yds. per foot of sewer. The invert was a single ring of brickwork, and the top was three rings, except across certain lands where a fourth ring was added. The cost per foot given in the table, $20 19, is for the three-ring section.

Section 2, Spillway.—The cost of pumping on the spillway was excessive, as the greater part of the work was done in freezing weather, and the floor of the spillway was below water level, so that the pump had to be run night and day during a considerable delay in the delivery of material. Its total cost was $6 774 70. The throat of the spillway forms 35 ft. of the main sewer.

Section 3, 70 Ins. Diameter, 1 714 Ft. Long.—There was less excavation on this section than on Section 1, and it was nearly all good material to handle, but nearly all in water.

The section built was similar to Section 1, containing an average of 80% as much material per foot as the 94-in. section; 80% of $20 19 is $16 15, while the actual cost of this section was $16 51 per foot. This increased cost was nearly all accounted for by a general increase in prices of material which took place about November 1st, 1894.

Section 4, Levee, About 3 200 Ft. Long.—Nearly the whole of the extension from Twenty-sixth Street to Thirty-fifth Street is on ground which was once the bed of the river. Above Thirty-first Street this low ground had been partially reclaimed by a levee built by the landowners, but at and below this street the line was exposed to the river. To protect the sewer a levee was built. It was some 3 200 ft. long, about 10 ft. high, and with 2 to 1 and 3 to 1 slopes, with 8 ft. crown. The river slope was covered with slag, a trench 3 ft. deep and about 5 ft. wide being first dug at the toe of the slope. This slag averaged 3 ft. in thickness at the bottom and 1 ft. at the top. The cost per foot was about $3.

Section 5, 70 Ins. Diameter, 211 Ft. Long.—Near the lower end of Section 3 the excavation entered rock which promised to be solid and homogeneous, and Mr. Martin decided to dispense with two rings of brick in the invert, the concrete base and stone cradle being, of course, dropped to bring the excavation, made as near as possible to the proper section, to semi-circular form by concrete and then to put in one ring of brick for the invert. As seen by the cost per foot, $9 41, this was economical, but it was soon found that the rock was not uniform, being full of soft places. This form of cross-section was abandoned after 211 ft. were built.

Section 6, 70 Ins. Diameter, 503 Ft. Long.—Sections 6, 7 and 8 were built in inverse order, work beginning at the lower end of Section 8 on February 21st, and being completed to the upper end of Section 6 on May 10th, or 2 846 ft. of sewer was built in two and one-half months, nearly three weeks of which time was lost by injunction proceedings over right of way. The rock extended the whole length of Section 6, but was very soft in places, being hardly more than an indurated clay. The excavation averaged about 11 ft. in depth, running from 7 to 14 ft. Timbering was used over nearly all of the section. The cross-section was full three-ring work, all brick in the lower half being laid to a line. No water was encountered, and no drain pipe was used. The cost per foot was $10.816.

Section 7, 77 Ins. Diameter, 947 Ft. Long.—The rock extended the whole length of this section, varying in hardness as on Section 6, and was overlaid with loose sand, necessitating timbering over its whole length. The cut averaged 11½ ft. The cross-section was full three-ring work, and the cost per foot was $12.567.

Section 8, 77 Ins. Diameter, 1 396 Ft. Long.—There was no rock on this section, but a large amount of water in sand and gravel, principally the latter. The water was carried into a borrow pit and thence into the river by a side pipe at the lower end of the section. The cut averaged 12½ ft., and timbering was used the whole length. The cost per foot was $12.292.

Section 9, 77 Ins. Diameter, 1 094 Ft. Long.—The material here changed to fine loose sand. The average cut was 14 ft., going up to 17 ft., with considerable water. The line of the sewer on this section crossed four railroad tracks, the worst crossing being on an angle of 22° under the Burlington main track. This track was put on 12 x 12-

in. transverse timbers; the excavation was 12 ft. wide and 16 ft. deep in loose sand, and was kept open nearly two weeks, the sewer built and back-filled, and the track put back on the ground without bringing a regular train to a stop. The traffic, however, was rather light, there being only six or seven regular trains each way in 24 hours. Excavation was started May 5th, the first brick was laid May 10th, and the last June 6th, two days of the time between the two dates being lost by rain. The cost per foot was $12.956.

The average cost per foot of Sections 7, 8 and 9, all of the same cross-section, was $12 58. This includes the cost of all lumber bought for timbering and scaffolding, no allowance being made for the value of old lumber left when the work was completed.

Section 10, Wooden Box, 323 Ft. Long.—The lower end of Section 9 is connected with the river by a wooden box 3½ x 3½ ft. in the clear, laid on a heavy grade, the intention being to carry the brick sewer ultimately some 6 000 or 7 000 ft. further northeast, where the invert grade line will reach the surface and there may be a chance to sell the water for irrigation. The box was put in place before work on Section 9 commenced, and a 12-in. drain pipe laid under the box carried the ground-water away from Section 9. The excavation for this box was deep, averaging 16 ft., and about one-fourth was rock, clay marl, with much water. There is a reef of this clay marl, mingled with layers of soft sandstone, all along the river front from Thirty-fifth Street down. Sections 6 and 7 lie in the reef, but Sections 8 and 9 are back of it, and Section 10 cuts through it. This accounts for the large amount of water encountered so close to the river and at an elevation above its level, this reef acting as a dam to hold back the ground-water flowing from the south.

The following tables are self-explanatory when studied in connection with the description of each section.

TABLE No. 1.—Materials per Foot of Sewer.

Material.	Sec. 1.	Sec. 3.	Sec. 5.	Sec. 6.	Sec. 7.	Sec. 8.	Sec. 9.
	Cu. yds.	Cu. yds.	Cu. yds.	Cu. yds.	Cu. yds.	Cu. yds.	Cu. yds.
Excavation—Earth	3⅓	1¼	1.2	3.0	4.0	5.5	7.0
" Rock	.....	.....	1.5	1.4	1.4	.....	.....
Concrete	0.395	0.349	0.15	.....	.....	.....	.....
Stone masonry	1.8	1.25	.....	.....	.....	.....	.....
Brick "	0.753	0.588	0.583	0.885	0.967	0.967	0.967
Back filling	2.4	3.2	2.7	4.4	5.4	5.5	7.0

TABLE No. 2.—Cost per Foot of Main Sewer.

	Section 1. 94 ins. diameter. 2 393.7 ft.	Section 3 70 ins. diameter. 1 714 ft.	Section 5. 70 ins. diameter. 211 ft.	Section 6, 70 ins. diameter. 503 ft.	Section 7. 77 ins. diameter 947 ft.	Section 8 77 ins. diameter. 1 396 ft.	Section 9. 77 ins diameter. 1 094 ft.	Sections 7, 8 and 9. 77 ins. diameter. 3 437 ft.
Excavation	$ .891	$ .377	$1.236	$1.412	$2.058	$1.620	$1.766	$1.787
Pumping—Draining	.743	.595	....	.....	.078	.484	.282	.308
Concrete base	1.925	1.645	.635	.....	.....	.....	.....	.....
Stone cradle,	8.128	6.134	.....	.....	.....	.....	.....	.....
Brickwork	6.443	5.761	5.722	8.324	9.332	9.203	9.396	9.300
Back-filling	.832	.842	.347	.223	.357	.301	.822	.482
Engineering	.715	.663	.916	.572	.500	.463	.420	.460
Tools	.424	.320	·381	.150	.097	.100	.140	.112
Watchman, etc	.090	.178	.173	.134	.145	121	.130	.131
Total	$20.191	$16.515	$9.410	$10.815	$12.567	$12.292	$12.956	$12.58

TABLE No. 3.—Cost per Cubic Yard of Concrete.

Material.	Sec. 1. 946.6 Cu. Yds.		Sec. 3. 598 Cu. Yds.		Average, 1 544.6 Cu. Yds.	
	Quantity.	Cost.	Quantity.	Cost.	Quantity.	Cost.
Portland cement (K., B. & S.)	.756 bbl.	$2.61	.7 bbl.	$2.44	.732 bbl.	$2.543
Crushed rock	.777 cu. yd.	1.40	.72 cu. yd.	1.42	.754 cu yd.	1.409
Sand	.470 "	.165	.35 "	.125	.424 "	.148
Water	.....	.01	.....	.....	.....	.007
Labor	.....	.685	.....	.73	.....	.703
Total	.....	$4.87	....	$4.715	.....	$4.81

The cement was all Portland, and the proportions of the concrete 1, 3 and 6. If Louisville cement had been used in the proportion of 1, 2 and 5, this cost would have been decreased about $1 35 per cubic yard, making the average about $3 46. This does not include tools, pumping or engineering.

The broken stone was unscreened, crushed sandstone, paid for by the cubic yard, but on an agreed basis of 2 500 lbs. for a cubic yard, the car weights being taken. There was no dirt in it, but many small pieces, the largest being about the usual 2-in. ring size.

TABLE No. 4.—Cost per Cubic Yard of Stone Cradle.

Material.	Sec. 1. 4 295 Cu. Yds.		Sec. 3. 2 143 Cu. Yds.		Average. 6 438 Cu. Yds.	
	Quantity.	Cost.	Quantity.	Cost.	Quantity.	Cost.
Rubble stone	1.3 tons.	$1.965	1.29 tons.	$1.995	1.297	$1 975
Cement	.848 bbl.	1.175	.93 bbl.	1.43	.875	1.261
Sand	.292 cu. yd.	.115	.33 cu. yd.	.16	.305	.13
Water	.....	.005	.....	.005	.....	.005
Labor	.....	1.27	.....	1.315	.....	1.284
Total	.....	$4 53	.....	$4.905	.....	$4.655

This does not include pumping, tools or engineering. The stone was bought by the ton, car weights being taken. It was sandstone easily worked, and, as shown by the amount used per cubic yard, of a low specific gravity. It was admirably adapted to the work where great strength was not needed, but rather a stone working easily under the hammer. A little Portland cement was used for plastering the top of the spandrel wall during cold weather, but nearly all was Louisville cement bought in paper sacks, 265 lbs. to the barrel. The mortar was mixed 2 to 1.

TABLE No. 5.—Cost per Cubic Yard of Brickwork.

Material.	Section 1. 1 800 Cu. Yds.		Section 3. 1 009 Cu. Yds.		Section 5. 123 Cu. Yds.		Section 6. 445 Cu. Yds.		Sections 7, 8, 9. 3 325 Cu. Yds.		Average, 6 702 Cu. Yds.	
	Amount.	Cost.	Amount.	Cost.	Amount.	Cost.	Amount.	Cost.	Amount.	Cost.	Amount.	Cost.
Brick, number	438.3	$4.07	431	$4.89	439	$4.58	450	$4.71	439.6	$4.75	438.9	$4 584
Cement, barrels	.835	2.01	1	2.365	1.07	2.63	.87	1.71	.939	1.81	.919	1.953
Sand, cubic yards	.34	.15	.35	.175	.375	.19	.42	.21	.46	.23	.408	.198
Miscellaneous	.....	.31	.....	.365	.....	.17	.....	.085	.....	.16	.....	.229
Labor	.....	2.025	.....	1.985	.....	2.25	.....	2.695	.....	2.66	.....	2.384
Total	.....	$8.565	.....	$9.78	.....	$9.82	.....	$9.41	.....	$9.61	.....	$9.348

Portland cement mortar mixed 3 to 1 was used in laying the inner ring of brick on the whole work and for plastering on Section 3, where the top was nearly all put on during freezing weather. The

Louisville cement mortar used on the rest of the brickwork was mixed 2½ to 1 on Sections 1, 3 and 5, and 3 to 1 on Sections 6, 7, 8 and 9. If Louisville had been used in place of the Portland, the average cost per cubic yard of $9 35 would have been reduced to about $8 70. This price does not include tools, except centers and templets, pumping or engineering, but does include all manholes, both special and regular, including manhole steps, covers and rings.

TABLE No. 6.—Detail Cost of Brick Work on Sections 7, 8, 9.—Cost per Cubic Yard.

Material.	Section 7. 916 Cu. Yds.		Section 8. 1 349 Cu. Yds.		Section 9. 1 060 Cu. Yds.		Average, 3 325 Cu. Yds.	
	Amount.	Cost.	Amount.	Cost.	Amount.	Cost.	Amount.	Cost.
Brick, number	442	$4.81	440	$4.70	438	$4.765	439.6	$4.75
Cement, barrels	.937	1.835	.99	1.91	.876	1.655	.939	1.81
Sand, cubic yards	.484	.24	.44	.22	.47	.235	.46	.23
Miscellaneous	.....	.135	.....	235	.....	.09	.....	.16
Labor	.....	2.63	.....	2 465	.....	2.95	.....	2.66
Total	.....	$9.65	.....	$9.53	.....	$9.695	.....	$9.61

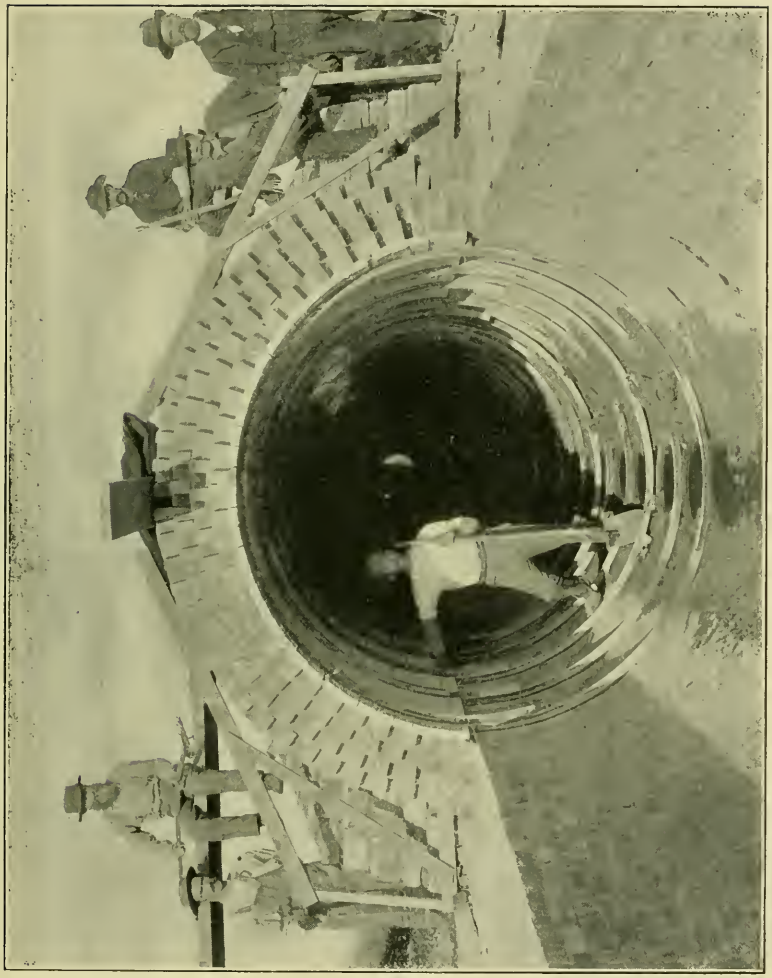
The miscellaneous item in Tables Nos. 5 and 6 is for slants, manhole covers, steps and rings, templets and centering and water.

TABLE No. 7.—Total Days' Labor Performed, including Levee and all Extra Work.

Occupation.	Wages paid.	Days' work.	Remarks.
Foreman	$3 33⅓ to $5 00	726	
Stone masons	$3 60	1 398	4.7 cubic yards rubble masonry average day's labor.
Brick "	4 00	1 491	2 080 brick laid, equal 4.7 cubic yards brick masonry per day.
Labor (including watchmen).	2 50	385	Watchmen, blacksmith and timbermen.
Labor	2 00	8 115	
Labor	1 75	7 628	
Water boys	$1 00 and $1 25	363	
Man and team	$3 50	2 150	
Engineers and pumpers	3 00	252	

All for an eight-hour day except pumpers, who were paid $3 for 10 hours' work, but generally worked 12 hours.

PLATE II.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXXV, No. 773.
FOLLETT ON COST OF DENVER SEWERS.



The appliances used were of the most ordinary kind; all concrete was mixed by hand and put in place with shovels and wheelbarrows; the brick and mortar were lowered by hand ropes into the deep trenches from platforms in the usual manner.

The wages paid to all classes of labor employed, both skilled and common, were about 40% above the average rates in force when the work commenced. It is beyond the scope of this paper to enter into a discussion of the motives which led the Board to set so high a scale of wages. Suffice to say, the wages shown were paid, and members can easily make the necessary allowances when comparing the cost of this work with that of contract work where different rates of compensation are in force.

Mr. Martin perfected a very neat steel ring used for centering. Plate II shows the construction pretty clearly. The ring consists of two semi-circles of about 8-lb. steel, a section being taken of a height equal to the thickness of the lagging to be used. The lower half is bent to form with the flange toward the center, its outer radius being that of the invert. The upper half is bent with the flange out, its radius being that of the lower half, less the thickness of the lagging. To each end of the lower half is riveted a short piece of the same section of rail with a hole punched in the flange, a corresponding hole being punched in each end of the upper half. One of these pieces has its upper end cut on a slight bevel, corresponding to a similar bevel on the end of the upper semi-circle. The two halves are joined by two lugs and bolts at each end. The lagging used was dressed out of 2 x 4-in. stuff with radial joints, and dressed on both sides. Each piece had fastened to its lower side three little iron clips, each one being held in place by two wood screws. The end of each clip was bent away from the wood just the thickness of the flange on the steel ring. It is frequently the case in perfecting a new appliance that some minor part of the device causes all the trouble. In this case the devising of a means to attach the lagging to the rings was the difficult point. Several plans were tried and abandoned, but finally this clip was suggested. It solved the problem and made the combination a complete success. Five rings were used to each 12-ft. length of lagging, so that forty rings sufficed to set up 96 ft. of centers.

In use, when ready to knock down a length of centering, the workmen removed the bolts from the pair of lugs at the bevel joint of a

ring, struck the latter a sharp blow with a hammer, and collapsed the joint. The three intermediate rings were first removed, then the end ones knocked out, and the lagging released. The collapsed rings were dragged forward through the 84 ft. of centering already in place, and set up again and bolted. The lagging was brought forward, thoroughly cleaned in the joints, and put in place, one piece at a time, a slight shove endwise engaging the clips on the rings, and an occasional sixpenny wire nail driven on the opposite side of the ring keeping the strips from working loose. The last piece of lagging wedged the whole section together, and the outside of the section was scraped clean with a shovel and wet down.

When building 96 ft. of top per day, two men would take down, clean and set up this centering, besides doing some cleaning in the finished sewer, making the cost of moving a little less than 4 cents per foot. In building the 8 290 ft. of sewer, three sets of rings and two sets of lagging were used, costing (including bolts, screws, clips and labor) about $775, or 9.3 cents per foot ; or the whole cost of centering, including moving, was a little over 13 cents per foot of sewer built. Of course, where the diameter of a sewer changes frequently, new rings would be required for each diameter, and the cost per foot increased. On this work the sizes were 70, 77 and 94 ins. only, and so but three sets of rings were used.

DISCUSSION.

HENRY GOLDMARK, M. Am. Soc. C. E.—In railway work the conditions are usually somewhat unlike those in city work, but the conclusions derived from experience are not very different. Contract work is more desirable and cheaper as a rule than work by the day, but there are two cases where the latter is preferable. One is when there are conditions which are not fully understood before beginning operations, so that the contractor is obliged to charge for a considerable element of risk and uncertainty. The second case occurs when an especially good grade of work is required, as this can be obtained more cheaply by day labor under the engineer's direct superintendence than it can under very good inspection, where it is done under ordinary contract conditions. Mr. Goldmark.

In one or two instances the speaker had found it advisable in every way to supply sand and cement in large quantities to the contractor and allow him to use it freely under some supervision, rather than to specify the amount of cement and sand and allow the measuring to be done with the idea of using the exact amount specified. On one occasion, when a number of abutments, piers and arched culverts were constructed on a railway, about half were built by contract and half by day labor, under the speaker's supervision. The cost of the work was nearly the same under both methods of construction, but the quality of the work done by day labor was decidedly better, although the contractor made but a very small profit on his work.

With regard to the high wages mentioned in the paper, the rates of $1 75 and $2 a day for common labor were exactly what were paid in Eastern Kansas and Southwestern Missouri on the railway work mentioned previously, although the railway company was paying only $1 10 for the ordinary track force. For $1 75 a day a gang of about twenty negroes was secured, who were exceptionally strong and vigorous men, and more than earned the extra amount they were paid per day by their superior efficiency in excavating heavy soil for foundations and in mixing concrete. The force account showed that the cost per cubic yard of excavation and of concrete was less than it had been under the contractor's method, employing men at $1 25 a day. This case shows forcibly that high-priced labor gives cheap results if it is carefully selected and carefully watched. It was the speaker's experience that where an engineer is not overburdened with work and is able to give the proper amount of time to a close daily supervision of the construction and has a perfect competent inspector on duty all the time, who can do essentially the work of a foreman, a high grade of masonry can be secured for a sum which would only give an average quality of work at contract prices.

Mr. Crowell. FOSTER CROWELL, M. Am. Soc. C. E.—The importance in all work, whether done by contractors or under the direction of engineers, of keeping a full and accurate force account of the details of the undertaking is obvious, and the paper gives an illustration of particularly careful supervision in this particular. The desirability of having work done by day labor under the direction of engineers or by contractors must depend upon the special conditions of each problem. There are some cities in which the contract system is the only one that should be adopted at the present time, but there are others where the work done by day labor would show a great saving and serve as an object lesson as to what can be accomplished at a reasonable cost. Any city where the same opportunities are given as in Denver could secure equally good results without any very great difficulty, and if the facilities for transporting material are more favorable the results will be better. Most cities began their public works by the day labor plan, but have been forced to adopt the contract system in self defence. The latter has improved in this country in many respects, and where the contractor has not been allowed to have too much latitude, the results are certainly excellent. Whether work is done by contract or not, the paper contains a valuable lesson to engineers who have this kind of work in charge, that they will obtain much better results by keeping this elaborate and troublesome kind of a force account. The results justify it, and the city will be the gainer, whether the accounting is only for the purpose of finding out what such an undertaking actually costs, or to show the citizens what are the detailed expenses of work performed properly.

Some years ago the speaker had under his supervision a number of contractors who were building a railway. One morning it was found that the contractors on seven or eight sections had abandoned the work and left the State. The railway company had either to relet the sections, which would have taken considerable time and trouble, as the work was in all stages of advancement, or to carry it on by day labor. The latter plan was adopted, and one of the first steps was to have the engineer corps previously employed see that the work was done properly. The plant left by the contractors was put in use, somewhat improved perhaps, but in general in the condition in which it had been abandoned. All members of the engineering party were instructed to keep an accurate account of what the men were performing. They made their entries from time to time during the day in their field books, and reported in connection with their engineering work; this force account was simply a record of what the men on different classes of work were performing. In a short time the work was systematized, and a few months later the books showed a considerable profit on the balance of the work on which the contractors had lost money. As construction progressed, the interest of the men in the

engineer corps continued to increase, and those in charge of different sections became quite emulous of one another in securing the best results. There was a very decided profit to the railway company arising from the interest which was awakened in that way. Mr. Crowell.

On other occasions the speaker had undertaken for a company work which could not be very clearly specified, owing to peculiar circumstances, and therefore could not be put under contract unless the contractor obtained a sufficiently high price to cover the risk. Sometimes the railway company preferred to take the risk itself, and probably saved money in the end; sometimes it lost money. In such cases it is extremely important for the engineer corps to have a knowledge of what men can accomplish in different classes of work, and what is a fair day's work under the particular conditions.

The speaker agreed with Mr. Goldmark, that there was often a real economy in paying high wages to secure good results. Men vary much in their capacity to do work as well as in their intention to do it, and men who feel they are well paid and know they depend on their performance to retain their positions will accomplish much more than those who receive simply the market price for labor. If the engineer has the authority to discharge men when they fail to do their work, there is generally no extravagance in paying a good price.

George R. Hardy, M. Am. Soc. C. E.—The advantage of doing work by contract lies in the fact that the contractor undertakes to accomplish the most work at the lowest price, and when he has named his price per unit for each class of work to be executed, the company or municipality has a clear conception of how much per unit the undertaking will cost, let the classes of work be more or less in amount. If unforeseen conditions arise, the company or municipality has to pay the cost of meeting them, for a contractor cannot be expected to gamble, so to speak, on whether or not there is rock in the excavation or the excavation will be unusually deep; he names a price at which he can probably carry out each item in the contract, while the corporation pays more or less than estimated for each class of work, according as the total quantities of each class exceed or fall below the estimates. Under this method of doing work, the duty of the engineer corps is confined to securing satisfactory results from the contractor. In this way the contractor has to bear the expense of forcing the men to work properly, and no expense is entailed upon the corporation on this account. For this reason the contract system is the best method of accomplishing large engineering undertakings where the magnitude of the work is so great that the engineer cannot personally supervise its execution. Mr. Hardy.

Accounting by a regular system for all expenses is a matter all engineers desire to investigate thoroughly, but accounting can be carried so far as to become too expensive. In the work mentioned in the

Mr. Hardy. paper the expense of accounting and engineering seems to have been combined, for 50 cents, 75 cents or 71 cents is given as the cost of engineering. If the engineering includes accounting, and there is a large force of clerks on the work so as to carry out the accounts in considerable detail, it will soon be found that the cost of this clerical work will far exceed its proportional advantage. If the work is done under an engineer able to see the relative importance of all the details, he will be able to obtain from day to day the reports necessary for a proper distribution of the total expenditure.

CORRESPONDENCE.

Mr. Landreth. WILLIAM B. LANDRETH, M. Am. Soc. C. E.—The paper is valuable as furnishing a means of comparing the cost of sewer work built by day labor in an inland city with that done by contract. Tables Nos. 3, 4 and 5 are of interest as showing the quantities of material and labor in each unit of finished work, and ought to be useful to persons estimating on such work. The cost of some of the material, especially the brick, is greater than the ruling prices east of Chicago, as would be expected, and the paper explains the reason for the excessive cost of the labor. The following statement, compiled from the tables in the paper, shows the cost per cubic yard of excavation, pumping and back-filling on the several sections.

Section	1	3	5	6	7	8	9
Cost	$0.734	$1.451	$0.541	$0.372	$0.462	$0.437	$0.410.

The paper shows that the cost for Section 1 was increased by the excessive quantity of water encountered, but the reason for the high cost of Section 3, $1.451, is not clearly stated. The cost of Sections 7, 8 and 9 indicates that the rock was not hard, as is stated in the paper. On sewer work done by contract in several eastern cities and towns under the writer's direction, the average cost for excavation, pumping and back-filling has been 30 cents per cubic yard. The work was on both pipe and brick sewers in earth and soft rock, with cuts from 6 to 20 ft. deep, and where the contractor made a profit. On other sewer work where the contractor bid an average of 19 cents per yard and abandoned the contract when the cuts reached 16 ft. to 18 ft., the municipality completed the work by day labor at a cost for excavation, pumping and back-filling of 63 cents per yard. The work was in loose gravel, with water and quicksand in places, and was carried on with the same men the contractor had employed. The work cost the contractor as much as it did the municipality, and the case is cited

only as an instance of a contractor naming a price much below actual cost. Mr. Landreth.

The writer's experience has been that sewer work generally costs a city less by contract than by day labor. A study of the number of days' labor employed on the Denver work indicates that the force was well handled, and the amount of labor per unit of completed work was a fair average of that on similar work in other places. The question as to whether the Denver work could not have been done for less money with some form of a trench machine suggests itself, but a correct answer probably depends upon some local conditions not stated in the paper. The paper indicates that a permanent, solid construction was desired, and the evident care with which the work was looked after by the engineer probably secured that result.

Andrew Rosewater, M. Am. Soc. C. E.—Having had occasion to become acquainted with the features of the Delgany sewer through an engagement to report upon the original project, the writer visited the work twice during its construction and can say that it is one of the best pieces of mechanical sewer construction he ever saw. The iron rails supporting the lagging on the arch were very convenient and simple, and, as they were handled, there were no connecting lines of sectional work to be seen on the entire line of the sewer. The brick used was exceptionally good and the courses were laid to perfect lines throughout; this was to be expected to a certain extent on work done by day labor. The marked feature in the management was the absence of incompetent mechanics and shiftless workmen, which was due to the fact that the board, in its desire to demonstrate the practicability of doing work better and cheaper by day labor than by contract, placed the construction in the hands of engineer experts with exclusive authority to employ and discharge men, and this authority was rigidly exercised, according to the writer's observation. The tabulated results as to cost do not show any striking gain over that of contract work in this case, but assuming the cost to be the same as under the contract system, the material and workmanship are decidedly better than under that system. This is one of a few instances where experiments of this kind have been successful. In probably seven cases out of ten the political tendencies of boards made up wholly of scheming politicians to give sinecures to political hangers on, would have largely increased the cost of the work without securing any compensating benefits in quality of material or workmanship. Mr. Rosewater

L. J. Le Conte, M. Am. Soc. C. E.—The actual cost of any piece of engineering work depends largely upon local circumstances, and it is hard to draw comparisons which will have much weight between the day labor and contract systems. If a city happens to have a competent engineer with experience in the line of work contemplated, and the plant required to execute the work is small and inexpensive, then Mr. Le Conte.

Mr. Le Conte. there is no doubt that the city can save possibly 25% on the cost of the work by hiring the necessary labor, purchasing the materials and doing the work itself. On the contrary, if the plant required to do the proposed work is expensive and of a permanent nature, it is generally more economical for the city to let the work out by contract, due regard being given to developing a healthy competition. The chief reason for adopting contract work is to avoid keeping an expensive plant on hand after the completion of the work, which experience shows to represent so much dead capital, the interest on which increases rapidly. Any attempt to sell the plant always ends in a disastrous sacrifice. The actual cost of concrete on this sewer is given as $4 81 per cubic yard in place, which is certainly creditable for hand-mixed concrete. The actual cost of coursed rubble masonry side-walls and invert, $4 66 per cubic yard in place, is surprisingly cheap and hard to understand. In this class of work rubble masonry will generally cost much more than concrete. The author gives the cost of arch brickwork as $9 34 and $9 61 per cubic yard in place. This stands out in bold contrast to the cost of other work, and naturally suggests the propriety of using concrete for the arches and confining brickwork to the inverts where scour is likely to take place. Inasmuch as there were 10 000 cu. yds. of brickwork and only 1 545 cu. yds. of concrete used, the possibility for further economy seems to be worthy of consideration.

Mr. Nelles. G. T. Nelles, M. Am. Soc. C. E.—The author gives a very complete analysis of the cost of his work, and makes a good showing for the day-work system. In considering his results it should be borne in mind that this work was not done under the conditions that usually exist on city work, but under intelligent engineering supervision, free from political influences. It was in fact a test piece of work, for it was only after a long and bitter contention between the Board of Public Works and the City Council of Denver that it was decided to do the work in this manner. It is not necessary here to go into the reasons that prompted the board to do the work by the day, but it is sufficient to say that it had ample political reasons for the step. Having in the face of great opposition decided to build the sewer by hired labor, the board realized that it was necessary to make a record, and also to redeem the promises made in its arguments during the discussion of the matter. It went about the accomplishment of this object in a manner quite unusual with political bodies, and from the start recognized the fact that its wishes could only be attained through its chief engineer. Acting under his advice it employed an intelligent superintendent, an engineer experienced in this particular class of work, and gave him almost absolute authority in all matters pertaining to the construction of the sewer. By this action it was possible for the superintendent to do the work in practically the same manner, and at about the same cost that it could have been done by contract.

Under such conditions public work can be done at a minimum cost. The trouble will not be so much in doing the work, as in securing proper conditions. Another important factor in the cost of work done under proper supervision in this manner by cities, is the fact that they do not enter into a binding contract with themselves to do the work in a fixed manner, and under rigid specifications, as is the case when work is done by contract. On the contrary they are always at liberty to make such change in methods or materials as experience may prove to be beneficial and economical to the work. Under the contract system it is rarely possible to make such changes, no matter how desirable they may be, without raising a cry of fraud or violating some of the terms of the contract. As a consequence whenever there is a choice of materials or methods under the contract system, the most expensive to the contractor is usually adopted. Mr. Nelles.

The author states that the wages paid were high and that due allowance should be made for this in using his figures. It is the writer's experience that within reasonable limits nothing is lost by paying good wages. This is particularly the case where work is scarce, and where employment is due to merit rather than influence. An examination of the scale of wages paid on this work shows that common labor was paid more and skilled labor less than the current rates in Denver one year previous to the construction of the sewer. It would naturally be expected that this would increase the cost of work done by common labor and decrease the cost of that done by skilled labor, but it is shown by a comparison with the cost of similar work in the same locality, done under the writer's supervision and under a different scale of wages, that the reverse is true, and that the amount of work done per man varies in about the same proportion as the wages paid.

It is the writer's opinion that in most cities public work can be done to better advantage and at less cost by contract than by hired labor, and that while the author's work is a good example of the latter system, his results are far more favorable than can usually be obtained.

The form of centers used for constructing the arch is in many ways a great improvement over the centers in common use. They are light and easy to move and set up, and leave the interior of the sewer free from obstructions during the construction of the arch. The cost of this form of centers seems to be excessive and would make their use prohibitive on sewers of shorter lengths and varying sizes.

W. W. Follett, M. Am. Soc. C. E.—The work of keeping the detailed accounts was not onerous. The executive force on the work consisted of the engineer in charge, one assistant engineer, three rodmen, one bookkeeper and two to four foremen. Prior to January 1st, 1895, there was also a superintendent of supplies and a cement inspector, but on that date both were discharged. The bookkeeper kept the general time-book, and made up the pay rolls and pay checks, Mr. Follett.

Mr. Follett. in addition to keeping the regular book accounts of supplies furnished. A laborer acted as brick inspector under the direct supervision of the engineer in charge, and reported to the bookkeeper each night the distribution by sections of the day's delivery. The foremen stated on their daily reports the amount of cement and sand used on each section. The engineer in charge usually took the first Sunday of each month on which to make up the distribution sheets for the previous month, and he consolidated the monthly sheets of each section into the final one on stormy days or evenings, so that the system of distribution did not cost the city anything over the usual cost of supervision of similar work. The cost per foot for engineering varied considerably on the different sections, because the progress of the work was not equally rapid on all sections, while the cost of engineering, being the salaries of the engineer corps and bookkeeper, was practically constant.

It was not the intention of the author to intimate that the wages paid were too high. This work was started when there was no demand for labor and wages were badly demoralized. Mr. Nelles is in error in saying that skilled labor was paid less than it was one year previous to the construction of this sewer extension. During the summer of 1893 there was absolutely no demand for skilled labor. He probably had in mind two years before, when sewer bricklayers were paid $6 to $7, and stone masons $4 per day. The bricklayers did not lay as many brick per man on this work as they had formerly done when working for contractors, not because they were paid less for their labor, but because they were required to lay the brick carefully, as stated by Mr. Rosewater. While they had received $6 per day two years before, the wages for bricklayers elsewhere in the city at the time this work was under way were about $2 50. The bricklayers were receiving as large pay, and had the same incentive to work, as the laborers, and it is quite true that men will work better when paid good wages, especially when they know that time checks are filled out in the office on very slight provocation. By means of the discipline thus enforced one foreman handled 18 bricklayers, divided into three gangs, the total number of men on his force, including helpers and laborers, being 80. The foreman of stone masons handled about as many, while the trench foreman sometimes had 100 in his gang. After the pace had been set and the quality of work wanted was thoroughly understood by the men, there was no difficulty in getting standard work. The average mechanic prefers to do good work.

In this connection the author wishes to call Mr. Hardy's attention to the fact that the foremen on this work were only about half as many in number as the inspectors would have been had the work been done by contract, while the other executive force was the same, with the addition of a bookkeeper.

The rock in the trench was not hard, except in a few places. As stated in the description of Sections 6 and 10, it was very soft. Mr. Landreth is mistaken in his statement in that respect, and if he will examine Table No. 1, he will also see that there was no rock on Sections 8 and 9, as stated in his correspondence. Mr. Follett

With reference to the cost of excavation, pumping and back-filling, while in deep trench work it may be quite appropriate to lump these three items and charge them all to the cost per cubic yard of excavation, on work like that under consideration, this process is misleading. For instance, on Section 3 there was but 1¼ cu. yds. excavation per foot, or, the excavation being 15 ft. wide, the average cut was about 2.2 ft., nearly all of which was in water. A 12-in. drain pipe, costing 31 cents per foot, was carried over this section. The back-filling amounted to 2.4 cu. yds. per foot, nearly all of which was hauled either by wheelbarrows, wagons, or wheel scrapers, the earth thrown out of the trench being left where it fell, to form the haunches of the back-filling. It is manifestly improper to add the cost of all of these items together and divide the sum by the amount of excavation, as Mr. Landreth has done.

The principal reason why the rubble masonry cost less than the concrete is that the former was made with Louisville cement, while Portland was used in the latter. Had Portland been used in the rubble, its cost would have been about $5 90 or $6 a yard. The stone used broke square in the quarry, so that very little hammering was required in the trench, while the foreman in charge had a peculiar ability for handling material with few laborers. These conditions account for the low cost of the rubble masonry.

The question of substituting concrete for brickwork in the arch of a sewer, broached by Mr. Le Conte, is one admitting of much argument on both sides, and the author will not attempt to discuss it here any further than to say that the cost per cubic yard of concrete in an arch would be considerably more than in a flat bed.

Two factors enter into the consideration of the question of whether a city should do work by contract or by day labor, viz., the probable relative cost of the two methods, and the quality of work likely to result from each. The average contractor is in the business to make a profit, while frequently his work is handled by an engineer, the same as it should be if the city did the work. Unless the work requires an expensive plant, it seems reasonable to believe that the city, under the same management, can save at least a portion of the contractor's prospective profit. About two years prior to the beginning of the extension under consideration, the city had built by contract that portion of the Delgany sewer above Twenty-sixth Street. The 1 400 ft. immediately above Twenty-sixth Street is 92 ins. in diameter, and has the same cross-section as was used on Section 1. When the latter section was completed, the author made a very careful and elaborate analysis of the

Mr. Follett. cost to the city of the two prices of work, and he found that when due allowance was made for the difference in cost of labor and material, Section 1 would have cost the city some $26 50 per foot, including inspection and supervision, if it had been let by contract on a basis of the contract prices in force above Twenty-sixth Street, as against its actual cost of $20 19. This indicates a saving of about 25% resulting from the day labor plan. In the case under consideration the contractor has pending in the courts a claim against the city, a quite frequent aftermath of contract work, a portion of which he will eventually collect, and this will add materially, probably $2 or more, to the cost per foot of the contract work.

Every engineer appreciates the extreme difficulty of forcing the average contractor to comply fully with the specifications when so good an opportunity offers for successful scamping as there does in sewer work. Mr. Rosewater has mentioned the quality of work done on this extension. It seems to the author that it must always be much easier to obtain good work by day labor than by contract. Hence, even if no saving in cost results to the city, the better quality of work obtainable should incline city authorities to the day labor plan. The author does not understand why, as stated by Mr. Nelles, the results as to cost here given are better than can usually be obtained. Any body of city officials can, if it chooses, divorce politics from business, and make the construction of the public works under its charge a strictly business proposition, as was done on the Delgany sewer. Men competent to handle work can be found by city authorities as easily as by contractors, and the city can go into the market and buy materials about as cheaply as can the contracting fraternity.

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No. 774.

TESTS OF FIRE-PROOF FLOORING MATERIAL.—DISCUSSION AND CORRESPONDENCE ON PAPER No. 769.*

By HOWARD CONSTABLE, R. W. LESLEY, CALVIN TOMKINS, OSCAR LOWINSON, GEORGE A. JUST, M. J. BUTLER, J. R. WORCESTER, FR. VON EMPERGER, AND GEORGE HILL.

HOWARD CONSTABLE, M. Am. Soc. C. E.—While the author was working with a center-bearing machine to devise a fair test for the filling between iron beams for high buildings, the speaker was experimenting in the direction of uniformly distributed loads. He found that most of the tests made by the manufacturers were carried out by bricks piled over the surface and well bonded, so as to arch over and either not let their full weight come on the floor, or else produce merely a shearing strain. The speaker tried steel billets, and fairly good results were obtained by carefully piling them parallel with the beams and leaving a space over the center of the floor to allow for deflection, as there is always a settlement while the parts of the arch come to bearing and some deflection as it is loaded up to the point of fracture. The machine now used by him consists of tubes about 15 ft. in height and exactly 1 sq. ft. in cross-section. Mr. Constable.

* "Tests of Fire-Proof Flooring Material," by George Hill, Assoc. M. Am. Soc. C. E., *Transactions*, Vol. XXXIV, p. 542.

Mr. Constable. Water is run into the tubes, and the pressure due to it is measured by noting the height at which it stands.

The experiments made by the speaker, like those described in the paper, show that the weakest point in tile floors is at the skewback, and that it is important to specify a reinforcing rib coming above and back of the edge of the flange of the skewback beam. When the rib is lower, the arch is destroyed quickly under low total loads.

By separating the area tested in a floor from the adjoining portion by cutting openings across the arch from one skewback beam to the other, the results do not indicate the full strength of the floor.

A floor 13 ft. long and 8 ft. wide which the speaker tested recently was designed for a tenement house where the legal live load is 70 lbs. per square foot. The floor was 4 ins. thick, filled with German Portland cement and screened cinders, and was carried by 4-in. beams running longitudinally 2 ft. apart, and with 3-in. angles in the mid-span 4 ft. 4 ins. apart, riveted vertically to the 4-in. floor beams. This floor had a strength of 350 lbs. per square foot, corresponding to a factor of safety of five. Where vertical angle irons were not used the factor was about two, and where a 2 x 2-ft. section of the concrete was cut off by openings from the remainder of the arch, the factor was reduced to one and one-half. This shows that there is a great difference in the strength of floors, depending upon the extent to which the section is confined longitudinally as well as transversely. In making tests of this nature it is desirable to use loads agreeing with those to be actually carried. In one case the speaker employed a test load of 1 800 lbs. per square foot, which was beyond the strength of the beams supporting the filling and made shoring necessary. Such a floor has a needless factor of safety.

The reason for the use of such an apparently crude method of construction from an engineer's standpoint is to be found in the desire of most people for flat ceilings. The tile flat arch enables a level ceiling to be obtained by simply plastering the underside of the arch.

Large channels in a floor, which may serve as flues under certain conditions in case of fire, promote intense combustion. In a large building recently burned in New York City, two-thirds of the height of the floor beams was entirely naked and on top of the beams there was a flooring of two thicknesses of flooring boards. Consequently there was a large channel or flue between each two beams, and these

channels formed flues which prevented the flooring system from being fire-proof in the present sense of the word. Mr. Constable.

Cement floors and rock-plaster partitions after a fire and after water has been thrown on them are often covered with small pits as if the water had slacked the lime in them. The appearance indicates that the lime may have been burned and then washed away, leaving the sand without a binding material.

R. W. Lesley, Assoc. Am. Soc. C. E.—The experiments described in the paper show that concrete floors are much stronger than those of hollow tile, a conclusion confirmed by many tests made by the speaker in Philadelphia. The Melan arch, in which the concrete is reinforced by iron or steel beams, is merely one particular type of concrete iron construction; the Monier arch, consisting of a net of wire imbedded in concrete, and the Bordenave arch, much used in France in buildings and fortifications, are other types. Possibly the most interesting example of the last type is the 72-in. conduit of the Venice Water-Works, which is made of concrete strengthened with longitudinal rods of I-beam shape, wrapped about with spiral rods. Mr. Lesley.

The Fire Department of New York has raised the point that when concrete has been exposed to fire, and water is then poured on it, it seems to disintegrate into a mass of mud. The floors usually laid in New York in ordinary buildings are made of natural cement, common building sand, and broken limestone. The cement will show more or less carbonic acid gas by reason of the low heat at which it is calcined, the sand probably contains organic matter, and the limestone, if put into a kiln, will become lime, whether the kiln be a 10-story building or a manufacturer's furnace. This naturally suggests that in the construction of a concrete floor intended to resist fire, the materials employed should themselves have been subjected to fire at a high heat. In other words, to a base of Portland cement, formed by combustion and calcination to incipient vitrification, there should be added a silicious sand or gravel, free from organic matter, and broken trap rock, which is formed by fire. In the case of a recent fire in Philadelphia, where the entire ground story of a building was in flames, the cement floor above withstood the fire, although on the same day the tile floors of a building at the corner of Broadway and Bleecker Street, in New York, were destroyed in a fire. At the time of the great fire in Chicago one of a few buildings that remained standing in the burned district

Mr. Lesley. was an artificial stone structure built by the Freer Artificial Stone Company.

Mr. Tomkins. CALVIN TOMKINS, Assoc. Am. Soc. C. E.—The speaker asked if any instances could be cited of buildings in New York City, having limestone concrete floors, where the limestone had been converted into oxide of lime by heat during fires in the buildings.

Regarding the use of crushed limestone in concrete as compared with silica gravel and crushed trap rock, he thought it might reasonably be said, that, so far as any difference exists, it is in favor of limestone, since this is cheaper than the other materials, and for the additional reason that the effect of extreme heat on limestone is to convert the carbonate of lime into the oxide of lime, and in this process heat is used up. This chemical change begins on the outside of the particles of stone and proceeds very slowly toward the interior; it is only at a very high temperature that any chemical change is effected at all. The fire-proof qualities of paris green are accounted for in a similar way, the water of crystallization being driven off by heat which is used up in the process of performing chemical work. A limestone aggregate in concrete for arch construction and for other purposes has always been, and is now, the principal aggregate used in New York City, and unless some decided failures due to its use can be proved, it seemed unfortunate to interfere on theoretical grounds with established custom, and thereby entail needless expense on building operations.

Mr. Lowinson. OSCAR LOWINSON, Jun. Am. Soc. C. E.—In respect to manufacturing buildings, there is a question as to the power of a floor-arch to withstand impact, such as the falling of a case weighing from 600 to 1 200 lbs. from a height of several feet. The speaker had never succeeded in finding sufficient records of floor arches failing under impacts to determine beforehand the strength in the arch, and the information on the subject is meager and vague. Probably the reason for the lack of failures of this sort is shown by the large static loads which tests like those described in the paper show that the arches sustain. The speaker believed that in manufacturing buildings and warehouses, the arch should be regarded principally as fire-proofing, secondary reliance only being placed on its strength. His practice was to halve to the beams the sleepers carrying the floor, as this construction does not take up more floor space. The only objection to such construction

is the formation of air spaces, which is of secondary importance in this instance, because the floor itself would burn in case of fire. At the same time the surest test of strength in an arch will always be its power to withstand the strains coming on it, and he believed that tests should be made with that end in view. Mr. Lowinson.

George A. Just, M. Am. Soc. C. E.—The tests made by the author emphasize what earlier tests in this direction have shown, viz., that the weakness of the hollow tile side-construction arch exists in the skewbacks and in the horizontal webs of the blocks adjoining the skews, and that the end-construction arch when properly laid is much stronger than the side-construction arch. Mr. Just.

The adoption of any system of floor arching generally depends on such practical considerations as the necessity for flat ceilings, facility in construction, cost or fire-proof qualities. The object is to get the cheapest arch that will do the work. Patentees and manufacturers like to demonstrate the superior strength of their own arches, but as in all other structural work, it is only necessary that the arch does its work with safety.

It would seem from the present and some older tests, that floor arches, whether old-style common brick, concrete, Melan, metropolitan or end-construction hollow tile, meeting the requirements generally found in buildings, can be laid to sustain an ultimate load of 2 000 lbs. per square foot of arch. The old hollow tile side construction, however, even under the most favorable conditions, seems hardly to reach half this strength.

The live loads fixed by the present New York Building Law only range from 70 lbs. in dwellings to "150 lbs. and upwards" for factories and warehouses. Hence any of these systems can be used with safety in ordinary building operations, and the selection of any one will be from considerations other than strength. The prices too, under severe competition, have become very much alike for the different systems.

In his experience, the speaker had never observed a failure in floor arches that could be charged to the system used. Failures could generally be traced to careless laying, such as applying the mortar only to the edge of the blocks; to the weakness of the channel irons usually placed along the walls, which, when with little superincumbent weight, are sheared off and moved horizontally; or to an insufficiency of tie-

Mr. Just. rods; to placing the tie rods too high on the web; or to the bending or removal of the rods by the mason, who often looks upon them as more ornamental than useful, and as interfering with the easy execution of his part of the work. Of course in special cases, high ultimate strength becomes an important consideration. In his practice he had met cases in breweries where the live and dead loads amounted to almost 1 000 lbs. per square foot of floor. In such cases the old common brick row-lock arch would be found stronger and as economical as any of the more modern systems.

CORRESPONDENCE.

Mr. Butler. M. J. Butler, M. Am. Soc. C. E.—The author has not dealt with the question of the relative fire-resisting qualities of different materials. Terra cotta has had a most extensive and searching series of tests, some voluntary, others involuntary. Of the former, the best are those recorded in the *American Architect* for March 28th, 1891, and of the latter, probably the most severe ever given any material, the great fire in the Athletic Club Building in Chicago, in 1894. All the floors in the building were of porous terra cotta, and the entire structure, where protected, was unharmed.

Some recent tests made under the observation of the writer show that concrete will not stand the harsh conditions incidental to fire. If the cement has not dried out thoroughly, on exposure to a high temperature it will crack and crumble into pieces, or if thoroughly dried out and then subjected to a high temperature it will crack and crumble to pieces if drenched with water while hot. If this be the rule, what avail is it that the floors be strong; they will go down just as easily as though of wood. More light is required on this point, and extensive and authoritative experiments are needed before concrete can be considered a fire-proof material.

Mr. Worcester. J. R. Worcester, M. Am. Soc. C. E.—While the paper does not give all the data which might be desired, it certainly sheds considerable light on the subject, and it is a question whether a few conclusions in addition to those of the author cannot be drawn from the results. The great lack of positive information by which to determine the proper style and thickness of flooring to be used in certain cases must have troubled many engineers, and it may be useful to work out

whatever reliable tests there are to such form that they can be easily used, although any results must of course be considered as only preliminary, and subject to alteration and improvement, as may be found necessary upon later information. Mr. Worcester.

With this object in view the writer has extended and rearranged certain tests given in the author's Table No. 3. Nothing has been done with the Melan arches, as Table No. 1 is in such form that it can be readily used, and, moreover, the combined steel and concrete is susceptible of such an infinite variety of modifications that any general classification would be impossible from the data at hand. With the terra cotta and clay arches, however, there are comparatively few types to deal with.

The arches built without mortar have not been included for two reasons; first, because the tests show great irregularity in the results, and second, because in practice it is almost always the case that mortar of some kind is used. In order to bring the results to the same basis for comparison, the ultimate load has been reduced to the equivalent load per square foot uniformly distributed on a 60-in. span, assuming that the rupture was caused by the bending moment, which, of course, was not always the case in the side-construction arches. In this reduction, in a few cases, it is not possible to tell from the paper precisely the length of arch over which the load was distributed, but it is believed that the errors are not large on this account.

TABLE OF EQUIVALENT UNIFORM BREAKING LOADS.

No. of Test.	Nominal depth.	Rise of arch.	Pounds per Square Foot.			Remarks.
			Porous terra cotta. End construction.	Hard material. End construction.	Hard Material. Side construction.	
91	6 ins.	3.5 ins.	.....	820	.....	
95	8	5	.....	1 360	.....	
96	8	5	1 520	.....	.....	
94	8	5	.....	.....	500	Ultimate strength too low, but did not crack at a low figure.
93	8	6	.....	.....	1 200	
100	8	7	1 630	.....	.....	
101	8	7	.....	1 780	.....	
92	9	6	.....	2 980	.....	Phenomenally high.
97	10	7.5	.....	.....	1 180	
98	10	8	.....	.....	1 240	
99	12	10	.....	.....	1 710	Classed with side construction because skewbacks were so made.

Mr. Worcester. From this arrangement of the table two points of interest are apparent; first, the porous terra cotta end-construction is of about the same strength as the hard material of similar form; second, the ultimate strength is approximately in proportion to the effective depth or rise of arch. Assuming this latter conclusion to be true, it is possible to arrive at a simple rule for proportioning the arches, for the load per square foot must be inversely proportional to the square of the span and directly proportional to the depth, or, expressed algebraically, $w = c\,d \div l^2$ where c is a constant to be determined experimentally.

To get at the proper factor of safety, it is interesting to observe that in Test No. 101 a slight crack was observed when the load had only reached 14% of the ultimate capacity. In many of the end-construction arches it is observed that the first crack is noticed very early, though the ultimate strength is great. In order to avoid cracks it is perhaps proper, therefore, to take for a working load not over one-eighth the least of the crushing loads for end construction, which would give a value of c in the above formula of 720, supposing d to be in inches and l in feet.

The following table, based upon this formula of $w = 720\,d \div l^2$, may be taken as giving safe loads per square foot for arches similar to those represented by the author's tests, though possibly the figures approach those at which slight cracks may be expected. So far as shearing strength is concerned, while the factor of safety may not be great in this respect for the short spans, in no case would the table allow a shear per square foot as great as the least under which any arch tested failed by shearing.

TABLE OF LIMITING LOADS PER SQUARE FOOT FOR TERRA COTTA AND CLAY ARCHES.

Nominal depth.	Effective depth.	Span of Arch or Distance Between Beams.					
		3 ft.	4 ft.	5 ft.	6 ft.	7 ft.	8 ft.
6 ins.	3.5 ins.	280 lbs.	154 lbs.	100 lbs.			
8	6	480	270	173	120 lbs.		
10	7.5	600	337	216	150	110 lbs.	
12	9	720	405	260	180	136	100 lbs.

Fr. von Emperger, Esq.—The tests of hollow tile floor arches without mortar joints are particularly valuable because they enable engineers who do not care to rely on the strength of mortar in such joints to ascertain what loads an arch will carry if the joints are badly made or the centers struck too early. The writer believes, however, that the results of some of the tests are open to question, this impression being received while witnessing Test No. 92, in which the load was distributed over nearly the whole span by planks. In such a case the center of the arch bends and relieves itself almost entirely, while the load is applied chiefly near the skewbacks. This can be seen in Plate X*, Figs. 2, 3 and 8, and still better in Fig. 8, page 557,* where cracks appear only in the skewbacks, in consequence of the load coming upon them alone. A comparison of Tests Nos. 92 and 95 gives rather conclusive evidence on this matter. Arch No. 92, having a span of 65 ins., had frozen mortar and was but 24 hours old when tested, yet it did not break at 2 545 lbs. per square foot. Arch No. 95, having a span of 60 ins., and built of the same style of blocks, with a covering of mortar over the top, was three months old when tested, yet it failed under a load of 1 534 lbs. per square foot (Table No. 3 gives 767 lbs.). The only apparent reason for such a difference is that the first arch was loaded over 95% of its area, and the weight was transmitted by the manner of loading to the I beams, while only 5% of the area of the second arch was loaded. The writer therefore believes that in studying the paper, attention should not be paid to such tests as were made with distributed loads. The results of the tests with center loads are given in the table on page 134, the only change made in the author's figures being to double the center loads mentioned in the paper, because an error is made in comparing distributed and center loads when the latter are not doubled. Mr. von Emperger.

The writer believes the last form given in the table to be the best, but, considering the forms now in the market, the end construction is to be preferred, mainly because the tests of this form with and without mortar joints showed considerable similarity in strength, while this was not the case with the side-construction blocks. The maximum live loads these floors are adapted to carry may be determined by the following rule devised by the writer: The sum of the maximum live load proposed and one-third of the dead load must not exceed one-eighth

* See *Transactions*, Vol. XXXIV.

Mr. von Emperger.

of the average strength as determined by at least three tests. None of these tests, however, should show a strength less than three-fourths of the average, or six times the sum of the live load and one-third the dead load. Under this rule end-construction tile arches with 8-in. blocks and 60-in. spans have a permissible live load of nearly 200 lbs. per square foot. The calculations on page 528 of Volume XXXIV of the *Transactions* must be corrected accordingly.

All Melan and concrete arches described in the first part of the paper were tested with center loads, and the writer believes that the loads per square foot given in the table in the paper should be doubled. The most apparent difference between the failure of Melan arches on

BREAKING LOADS OF HOLLOW TILE ARCHES.

Test number.	Construction.	Rise. Inches.	Span. Inches.	Breaking load. Pounds per square foot.	Joints.
1*.....	Side.	9	72	856	With mortar.
93.....	"	6	60	1 336	"
94.	"	5	60	762	"
109.....	"	5.5	60	600	Without mortar.
110.....	"	5.5	60	1 000	"
112.....	"	5.5	60	500	"
113.....	"	5.5	60	550	"
95.....	End.	5	60	1 534	With mortar.
96.....	"	5	60	1 714	"
100.....	"	7	60	1 838	"
101.....	"	7	60	1 994	"
102.....	"	8.75	60	1 478	"
103.....	"	9	60	2 380	"
107.....	"	9.5	60	1 202	"
108	"	9.25	60	1 446	"
2*.....	Side.	9	72	1 616	"

* See *Transactions*, Vol. XXXIV, p, 528.

the one hand and concrete and tile arches on the other hand is that the concrete of the former simply cracked, while the latter dropped to the ground. In the latter case the tie rods were entirely uninjured, while in the Melan arch tests they failed. Arch No. 81 failed through the rupture of a tie rod, as did Arch Nos. 82–83, although this fact is not recorded. In Arch No. 84 the tie rod was slack, and the writer desired to have the test omitted altogether as valueless. The tests of Arches Nos. 81 and 83 gave breaking loads of 3 334 and 6 520 lbs. per square foot (the table in the paper gives half these loads) and are very satisfactory, but do not give full justice to the arches.

In testing floor arches the load a which they are built to carry corresponds with a horizontal thrust h. The breaking load A or

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$8a$ produces a horizontal thrust $8h$. The tie rods usually made have a factor of safety of 4, so that their elastic limit is reached when the horizontal thrust becomes $2h$. If the load $8a$ is to be used in testing arches and the elastic limit of the metal in the ties is not to be exceeded, four ties must be provided for tests in which one only is commonly used. The tests of tile and plain concrete arches showed the reliability of the material employed because the tie rods remained intact, while on the other hand the rods employed in the Melan arch tests were entirely inadequate. In using flat iron ties bent around the lower flanges of the spandrel beams, the writer intended to make a connection where there would be no means of screwing the arch tight after it was built, and thus increasing its strength. The rods were of poor material and presumably bent cold, as they opened like the fingers of a hand; they were applied too low to give proper resistance to the thrust, and they were not tight enough from the start; in fact only tie rods with nuts and screws should be used. For these reasons the writer believes that the results of Tests No. 81 and 83 do not correspond with the breaking loads of the arches, although they may be very near them. The cracks which appeared were simply a consequence of the increase in the span caused by the stretching of the ties.

Arch No. 87, which had been loaded in Tests Nos. 78, 79 and 80, is the only one where neither arch nor tie rods were injured; it carried 3 890 lbs. per square foot. This amount is double that given in the paper, but as a matter of fact it is not large enough, as the load was applied eccentrically. The writer does not believe that this eccentricity will amount to much in the case of small spans.

A comparison of Test No. 77, made on a bent iron beam which failed under 14 000 lbs., and a plain concrete arch which failed completely under 31 250 lbs., leaving the tie rods uninjured, with Test No. 83, made on an arch which carried 58 750 lbs. until the tie rod failed, shows very well the use of the combination of iron and concrete. The condition of the concrete of Test No. 83, after the load was withdrawn, is described in the paper, and indicates that it is incorrect to speak of a failure in this concrete, in the full sense of the word. The iron ribs were taken out with a sledge and found to be without any permanent deflection.

The writer desires to draw attention to the eight-day test of Melan

Mr. von Emperger. arches, as the fact that they would carry any load at all after such a short time has been doubted. Such tests have not been made before, and are of importance as indicating the length of time centers must be left in place, and whether the arches can be loaded immediately after the centers are struck. They show that the arches are sufficiently safe after eight days to allow walking on them, and Test No. 80 indicates that they will carry any load that may come upon them during subsequent building operations. Taking the results of the tests and submitting them to the writer's rule given on page 133, it will be seen that the permissible live load for the light Melan arches is 500 lbs. per square foot, and that for the heavy type is over 1 000 lbs.

Mr. Hill. George Hill, Assoc. M. Am. Soc. C. E.—In closing the discussion, the author submits in the table on page 139 the results of an additional series of tests made recently at the Ireland Building, in New York City, of commercially constructed fire-proof arches under various methods of loading. The arches were in all cases 8-in. side-construction, hard tile protection skews of 50-in. span. Tests were made with concentrated loads applied centrally and eccentrically, and with distributed loads on entire bays and on sections cut from the bays. The distributed loads were applied by means of cribwork, consisting of 3 x 4-in. yellow pine sticks 1 in. apart in the clear, decreasing in length and giving a true condition of uniform loading, as was demonstrated by series of levels taken throughout an entire test, showing that the arch curved uniformly over its span. These tests show the marked influence of length on strength, the inappreciable influence of the tie rod as a separating member, its small value in intermediate spans where the skewback beams are framed at both ends, and the great value of filling as a means of distributing the load. They also serve to indicate how great must be the capacity of a testing machine to give its results positive value.

The machine described by Mr. Constable is one which the author thought of early when considering the question of design, and discarded for the following reasons: First, because of its very limited capacity; second, because in an arch of great flexibility or elasticity there would be danger of the tubes touching at the top, concentrating their weight on lines of the base instead of over their entire area; third, because they are liable to freeze in cold weather; fourth, be-

cause in failure there is the likelihood that the tubes will pass down into the ground, may be ruptured, may rupture their connections, starting the water everywhere, and must be lifted out; fourth, because the weight is transmitted beyond the skewback beams to their connections, thus straining the entire framing around them to an unknown amount. Mr. Hill.

The author considers the practice announced by Mr. Lowinson of disregarding the strength of the fire-proof arches is very far from being good engineering, since it is only necessary to employ a carefully designed form of floor arch, to have adequate strength without any additional expense, and to save the cost of the wooden construction.

The correspondence by Mr. Worcester and Mr. Von Emperger emphasizes the remark in the paper that the tests do not afford sufficient data from which to draw conclusions, since in one case the arches laid without mortar are specially referred to, while in the other case they are expressly thrown out. Disregarding the derivation, the table of limiting loads per square foot given by Mr. Worcester is reasonably correct. Its value, however, except under unusual conditions, is minimized by the fact that other considerations make it desirable to employ an arch that is as deep as the beam which serves as its skewback. The cracking mentioned is due to inequality of bearing, and does not affect the strength seriously.

Taking up Mr. Von Emperger's communication, the author would say that throughout the whole of test No. 92 the center of the arch received its full loading, and was in no case relieved. Referring to Plate X,* Figs. 2, 3 and 8, it will be observed that in Fig. 3 the blocking is bent. In Figs. 2 and 8 the arch had failed, but in both cases, immediately prior to failure, the load was uniformly distributed over the entire area of the arch. Referring to Fig. 8 of this plate, it will be evident from the horizontal direction of the cracks that the arch failed along true thrust lines, as was shown clearly in the blocks. Mr. Von Emperger did not see these tests, and so the error is explicable.

Referring to Test No. 92, the arch was loaded until the extension of the tie rods by horizontal thrust opened the bottom joint as described, and caused a spall the size of a thumb nail at the top. Had the conditions been as stated by Mr. Von Emperger, this could not have occurred. The reason for the great strength was that the freezing of the mortar

*Transactions, Vol. xxxiv, p. 565.

Mr. Hill. gave an absolutely homogeneous joint of great compressive strength, which developed to the fullest extent the strength of the blocks. Test No. 55 failed, not because of the concentrated load, but because of the porous terra cotta skewbacks, the failure in which began at 43% of the maximum load. The statement made concerning the character of the tie rods used in the Melan system tests is an error, as the broken pieces which the author has in his possession show a break square across below the bend. By reason of these inaccuracies, the author considers it unnecessary to discuss this correspondence further.

Comparing Table No. 3* with the table on the opposite page, a very wide difference between test and actual conditions will be noticed immediately, and the author therefore feels that he has accomplished the purpose for which the tests up to No. 114 were inaugurated when he emphasizes the three conclusions drawn at the close of the paper.

Coming to the commercial consideration of the arches, it is necessary to remember that wood in the floors is being gradually eliminated, and that within a short time it is probable that the universal practice will be to lay concrete immediately over the floor arches, covering them or not according to the uses to which the rooms are to be put. When this is done, the concrete being more expensive than the blocks, an arch block will be used, so as to fill the void to the top of the beam flange. All kinds of material are used in the arches for jointing, good, bad and indifferent mortar, frozen mortar or mud; and if it is practicable to fulfil all of the conditions required of the floor arches without additional expense, and secure in addition sufficient strength to be independent of the character of the mortar, engineers will be relieved of considerable anxiety. Voids between the top and bottom surface of the floor are undesirable if continuous, as they serve no useful purpose.

Taking up now the tests under practical conditions, it is safe to draw the following conclusions:

First.—The ordinary practice of shallow arches and cinder filling, using the side-construction protection skew, leaves an insufficient margin of strength for any purpose except that of office buildings.

Second.—The architect in specifying for the strength of floor arches should state all of the conditions under which the tests will be made,

**Transactions*, Vol. xxxiv, p. 567.

TESTS OF HARD TILE SIDE-CONSTRUCTION PROTECTION SKEW ARCHES, WITH ABOUT 4 INS. OF CINDER FILLING.

The span was 50 ins.; the rise, 5 ins.; the depth of skew, 8 ins., and the area of the tie rods 0.5 sq. in. in every case.

Test number.	Width, inches.	Arch area. Square feet.		Total load. Pounds.	Load per square foot.		Horizontal thrust. Pounds.	Shear per foot skew.	Load.	Deflection. Feet.	Remarks.
		Total.	Loaded.		Arch.	Loaded area.					
114....	264	90	0.73	9 750	.....	13 350	12 150	.....	Center.	.....	Whole length of section cracked } one bay.
115....	264	90	0.73	18 500	.....	25 360	23 150	.....	"	.....	Sheared near center of bay } one bay.
116....	264	90	0.73	11 900	.....	16 300	14 890	.....	"	0.039	Loaded block crushed. } one bay.
117....	264	90	0.73	10 000	.....	13 700	12 500	.....	"	0.025	East end } one bay.
118....	264	90	0.73	13 500	.....	18 500	16 880	.....	"	0.083	West end } one bay.
119....	264	90	0.73	14 500	.....	19 860	18 130	.....	"	0.12	Center } one bay.
120....	54	18.72	0.73	9 000	481	12 330	11 250	1 000	"	0.08	Cut section.
121....	48	16.64	0.73	10 000	602	13 700	12 500	1 250	"	0.036	"
122....	48	16.64	0.73	6 000	361	8 225	7 500	750	"	0.078	"
123....	264	90	10.55	20 000	.....	1 895	50 000	.....	Distributed.	0.034	"
124....	264	90	10.55	25 000	.....	2 366	62 500	.....	"	0.051	Section affected 6 ft. long.
125....	54	18.72	9.7	11 500	614	1 180	28 750	1 280	"	0.115	Open bays; side skews sheared.
126....	48	16.64	10.28	17 000	1 020	1 655	42 500	2 126	"	0.068	Cut section; center depressed.
127....	48	16.64	10.28	12 500	752	1 215	31 250	1 565	"	0.038	" "
128....	48	16.64	10 28	15 500	933	1 510	38 750	1 940	"	0.065	Skew failed.
129 ..	264	90	10.28	22 500	.....	2 190	56 210	.....	"	0.16	Skew sheared next open bay.
130....	264	90	10	15 000	.....	1 500	37 500	.....	"	0.018	
131....	264	90	0.73	8 000	.....	10 900	10 000	.....	Center.	0.026	Cinders removed.
132....	264	90	9.75	17 500	.....	1 800	43 800	.....	Distributed.	.....	" "
133....	48	16.64	10.28	9 500	570	925	23 775	1 188	"	0.08	" " Sheared at skews.
134....	48	16.64	0.73	4 100	246	5 620	5 130	513	Center.	0.33	" " Broke underneath.

NOTE.—The arches tested were similar to those shown in *Transactions*, Vol. XXXIV, p. 559, Fig. 11.

Mr. Hill. since the strength of an arch 5 ft. long is totally different from the strength of an arch built of the same blocks 15 ft. long, the strength of the arch when covered different from the strength of the arch without, and the strength of the concrete cover different from the strength of the cinder cover. The strength per square foot when loaded with a concentrated load is different from the strength per square foot with a uniformly distributed load, and the effect of a uniformly distributed load different under different conditions of distribution, the most satisfactory result being obtained where an elastic material like wood is employed under proper conditions to effect the distribution.

Third.—As a general lesson to be drawn from all of the tests, no test can be of full value that is not carried to the destruction of the arch.

Vol. XXXV. JULY, 1896.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
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TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 775.

BANK REVETMENT ON THE LOWER MISSISSIPPI.

By H. ST. L. COPPÉE, M. Am. Soc. C. E.

PRESENTED FEBRUARY 19TH, 1896.

WITH DISCUSSION.

The following paper is written for the purpose of giving in logical sequence the operations of engineers on the lower Mississippi River in dealing with the problem of bank protection, as derived from personal knowledge and experience and information on the subject gleaned from the scattered appendices in the reports of the Chief of Engineers of the United States Army. Much valuable engineering knowledge is buried in these executive documents that can be resurrected with great difficulty, as the different members of one subject have often been interred in many volumes, frequently covering a considerable range of years.

By the lower Mississippi is meant that portion of the river from Cairo to the mouth, which is, strictly speaking, an alluvial stream, its waters charged with sediment received from its tributaries and taken from its crumbling banks, the amount varying with its course from bend to bar. It flows through an alluvial plain, in great part formed by its own agency.

It is the purpose of the author to give a brief description of the different kinds of revetments used to protect the banks of the river from erosion, their cost, manner of construction, and the results obtained by their use. The relative value of different methods of river improvement, the success and cost of certain projects or lines of procedure as a whole, will not be discussed. The author will confine himself strictly to the details of construction and their evolution from the cruder methods of twenty years ago to the more advanced practice of the present day.

It is a well-known fact that revetment made of brush, cane, straw, etc., has been used in Europe and Asia for the protection of river banks and levees for many years, and it is therefore not a novelty; but the manner in which it is constructed differs very considerably, even in this country, varying from the log mattress of the sea wall to the closely woven fascine revetment of the non-tidal stream.

It is customary in protecting the banks of a river to use, if possible, the material found in most abundance and procured most economically in the immediate vicinity of the improvement. On the lower Mississippi the foreshores, bars and low-lying lands are covered to a great extent with an abundant growth (thousands of acres) of willow and cottonwood saplings that can be woven into mattresses with great facility, when cut and shipped to the working ground. Stone, though not so plentiful, is obtained at reasonable prices from the quarries of the bluff formation of the Mississippi, the foot-hills of the Ozarks on White River and other portions of Arkansas, and also from Alabama, but a few hours distant by rail from the lower river; and as ballast from vessels in the port of New Orleans. The other articles of which the revetment is manufactured—spikes, wire, cable, manilla rope, staples, etc.—are purchased generally in a northern market. The labor employed is procured from the floating white population of the country and the colored residents of the southern States bordering on the Mississippi.

The purpose of the revetment, as before stated, is to prevent the bank caving (which, without protection, is constantly occurring on the concave side of the river), and thus reduce the load of sediment carried to the bar, preventing its rapid growth; to maintain a normal width and depth in the bends, and to protect valuable property in danger of being destroyed by the constant action of the currents.

The difficulty of constructing and maintaining works of this character on the Mississippi River is very great, owing to the excessive variation in the height of water surface and the volume of discharge, the great depths, swift and changing currents in the caving bends, and the short season when the water is at a stage sufficiently low to admit of building and sinking mattresses.

The methods at first adopted on the lower Mississippi to keep the banks from caving were similar to those in use on the Missouri, which, though a smaller stream, is subject to much the same conditions of flow and bed. The first revetment used on the lower river was purely local and for the protection of property, and not with a view to bettering navigation or in any way improving the river for the purposes of transportation. Prior to 1878, as the river at Memphis had been encroaching on the shore in the vicinity of Wolf River and below, threatening the destruction of valuable business houses and other property, an appropriation was granted by Congress to be expended under the direction of a United States engineer officer, for the amelioration of these very serious conditions.

At Vicksburg, in the spring of 1876, a cut-off occurred, leaving the city on the bank of a lake, the west side of the new channel having rapidly caved away, thus carrying the navigable water farther and farther from the town wharves and landing. To check this action, on the recommendation of a board of United States engineer officers, Congress allotted funds for the promotion of a project of improvement having as an essential factor the prevention of the bank caving at Delta Point.

At New Orleans a project was formulated by the United States engineer officer in charge of that district for the protection of the wharves threatened by the scouring action of the water, the purpose being to build a timber bulkhead in about a line with the outer edge of the wharves, and also to carpet the slope below that line with mattresses. In 1878 money was obtained from Congress and the next year work was commenced. The methods employed by Captain Eads on the jetties at the mouth of the river will not be considered, as that work consisted in the building of new (artificial) banks rather than the protection of the natural ones, and also as the conditions directly at the gulf are so dissimilar to those existing above. The lower reach at the jetties partakes somewhat of the character of a tidal estuary, as

well as a sedimentary river, though no tidal inflow is perceptible at the surface.

The lower Mississippi River flows through a low-lying land subject to inundation, composed for the most part of sedimentary deposit formed by the river itself, though undoubtedly having its origin at some points at a period remotely antecedent to the fluvial existence. The material as exposed in the caving banks is found generally in horizontal strata of varying thickness and consistency, no definite order being maintained, the position of each layer having been determined by the special locality, and its location relative to the current of the river during its formation. A section of the bank at Delta Point above low water shows a thick top layer of soil; then buckshot; then sand; then a thin layer of clay and buckshot; then sand; then yellow clay; then sand, underlaid by a thick layer of blue clay reaching to low-water level. At some points the section is mainly sand; at others, the major portion is buckshot, the latter being a tenacious black clay, which, if continuous and not intersected by layers of sand or lighter material, would withstand the action of the water almost as successfully as rock. This indicates how impossible it would be to modify the form of revetment to suit each arrangement of strata, changes in which may occur in less than 100 ft. length of bank.

The object has been to endeavor to construct all revetment sufficiently strong to protect the weakest material. The position of the strata and the drainage of the back-lying lands are also potent factors in the problem of improvement by revetment, as the lighter strata, sand, etc., act as drains for the ground-water at low stages, causing much damage by sloughing.

The position and extent of the soft strata in the bank, the drainage of the adjacent land, the degree of curvature of the bend, the slope and velocity of the water, the position and depth of the channel, the kind of growth on the land, and the stage of the river, determine the extent, rapidity and kind of caving. Most of the bank caving occurs at medium high stages of river, and is specially extensive on a rapid decline after a continued high water. The channel of the river in the bends probably deepens or scours at most points during the high water and rising river, when the velocity is greatest; but the actual caving of the bank does not as a rule take place until the support of the water has been taken away.

At some points the bank caves at all stages, this action being caused principally by abrasion; while at others, the combined action of the seep or drainage water through the soil and the undermining of the foot of the slope causes what is termed sloughing. This kind of caving is most difficult to contend with, the bank often sliding out in great sections, carrying with it willows and stone and necessitating a re-adjustment of the work for some distance above and below. At the bank-full stage in many places the thread of maximum velocity occupies a position approaching more nearly the center of the stream, thus decreasing the scouring action at the sides.*

Vicksburg Harbor, 1878 to 1881.—The first work at Delta Point, opposite Vicksburg, consisted of the revetment of the bank below medium low water, it being expected that the upper bank would be graded to a gentle slope by the action of the water, not needing other protection.

The plant, labor and materials, with the exception of rock, used in the early construction was furnished by the Government, the rock

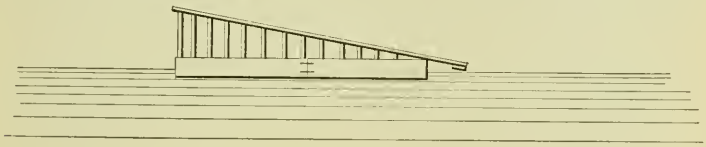


Fig. 1.

being contracted for. The protection work consisted of willow mattresses 50 ft. wide by 150 ft. long, constructed on floating ways. An attempt was made in November, 1878, to use stationary land ways, but they caved into the river, and the method was abandoned. The floating ways or barge on which the mattress was manufactured consisted of two coal boats, each 165 ft. long, with the gunwales cut down, placed parallel to each other and bolted together (see Fig. 1). On them the ways proper were erected, having inclined skids 6 ft. apart running across the barge, supported at intervals of 4 ft. by posts. The upper end of the skids or runs was 8 ft. above the gunwales, the lower end resting on the barge. This inclination proved too slight for starting the

* A recorded instance of bank caving occurred in Gouldsboro' Bend, New Orleans Harbor, December 14th, 1887. Just above a spur dike, a mass of earth in the shape of a semi-ellipse in plan, having an axis of 300 ft. and semi-axis of 60 ft., moved downward and outward into the river. The movement was regular and at the rate of about ½ in. per minute. The bank was quite steep before the caving. The cause was a pond on the bank from which the water seeped into the river.

mattress, and had to be increased. The runs extended 6 ft. beyond the edge of the barge, resting in the water when the weight of the mattress was on them. The timbers of the framework were 4 x 6-in. cypress, the runs being of long leaf pine. Stringers were fastened below the runs at the lower end, to strengthen them. On this barge a mattress 50 x 175 ft. could be constructed.

Willow brush and poles were obtained from a bar in near proximity to the work and placed on barges. When ready to manufacture the mat, these barges were towed alongside of the mattress ways, and the material taken from them as needed, the mat barge being fastened to the shore, and the mattress barges to the material barges by manilla rope. The laborers on the material barges passed the brush and poles to the mat-builders on the mat barge, as required. The mattress construction consisted in first placing longitudinal stringers made of willow or cottonwood poles 6 ins. in diameter at the large end, fastened

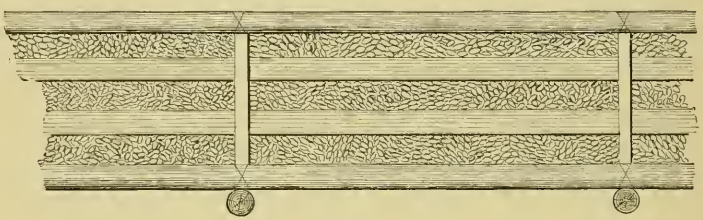


Fig. 2.

together with wire and spikes, 8 ft. apart on the runs, each stringer stretching along the barge the full length of the mattress. In a mat of 50 ft. width there were about seven stringers. Across these and on top of them were then placed a set of transverse stringers of the same dimensions, also 8 ft. apart, fastened to the longitudinal system at the intersections with wire and spikes. At the intersections were also inserted, in a vertical position, pins of swamp hickory 1 in. in diameter and 3 ft. long.

On top of this framework and between the pins were then laid five layers of small willow brush, each layer at right angles to the one above and below, until a thickness of 2 ft. was obtained (see Fig. 2). On this brush was placed a set of transverse stringers 8 ft. apart, in the same manner and of the same dimensions as those below. The upright pins were let into these stringers and secured by wedges and nails, thus connecting the upper and lower system of poles. No. 12 wire was

also used at the intersections to connect the top and bottom frames, adding strength to the bond.

When the mattress was completed, ropes were fastened to it in such a manner as to be readily freed by toggles when on the bottom. Then it was launched from the ways into the water and towed to the part of the bank to be revetted. A large coal boat bottom was used as a mooring barge, to which the mattress was fastened by lines, the mooring barge being held in position by shore lines (as shown in Fig. 3); the mat had independent lines to the shore, that could be released by trip toggles after sinking. When in position and ready for sinking, sufficient rock was thrown from the mooring barge and small stone flats, 30 ft. long by 12 ft. wide, to cause it to settle to the bottom. A steamboat was constantly employed in placing the barges, towing, etc.

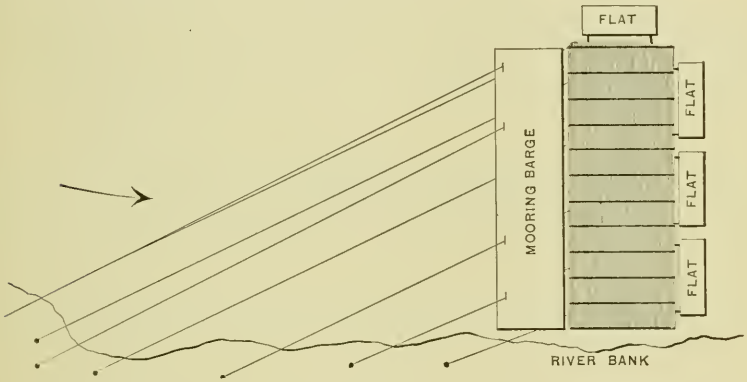


FIG. 3.

These first mattresses extended from about the low-water line 150 ft. down the subaqueous slope. One could be made and sunk in a day with the untrained labor and crude appliances, and under the unfavorable conditions of weather and river stages. In 1879 a great number of these mattresses were sunk successfully at Delta Point. The mat-builders became so proficient that as many as four mats were sometimes made and sunk in one day. A considerable reach of river front was thus revetted. In some instances, the depth of the water at the outer edge of the mat was 80 ft.

Experiments were then made with floating screens, similar to those used by Colonel Suter, U. S. A., on the Missouri, but they proved very unsatisfactory.

The local engineer gives the cost of revetment with these early

mats as about $18 per lineal foot of bank ($12 per square)* under favorable conditions. It must be remembered that this only represents subaqueous work, the upper bank not being improved in any manner. In 1880 it became apparent that the protection of the subaqueous bank only, leaving the upper slope exposed to the destructive action of the currents, was bad practice, and that some method would have to be adopted for its preservation. This was accomplished by grading the upper bank to a slope of about 45° with shovels, from about the low-water line to the top, and placing on this uniform slope a revetment composed of willow fascines laid to a thickness of about 8 ins., having on top transverse poles which were fastened to the ground by wires to deadmen, thus holding the willows in place. A deadman is a beam set horizontally in the ground forming an eye or loop as shown in Fig. 4. No rock was used on this revetment.

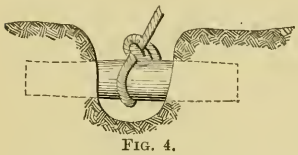


FIG. 4.

In 1881 it was proposed to institute a new method of bank grading which had been successfully applied on the Missouri, viz., the removal of the material of the upper bank by means of a jet of water. For that purpose a fire pump was placed on a barge 16 ft. wide by 98 ft. long by 3½ ft. deep, the pump being of the Dayton cam make, with cylinders 16½ x 18 ins., and water plungers 9 ins. in diameter, and with two 4-in. discharge pipes. On the barge was also a boiler 42 ins. in diameter by 20 ft. long, with two 14-in. flues. The total cost of the grader complete was $3 679. This machine was not used during the season of 1881, owing to adverse conditions of river stage, etc., the work, as before, being confined to mat-building; the method of mat construction was entirely changed, the pole frames at the top and bottom and the hickory pins, which were found to be weak, being entirely dispensed with, and in their place a wire netting substituted, fastened by wire stitches made through the mat.

Before giving the details of construction, it will be well to describe the mattress boat on which this form of revetment was built. It consisted of two barges 20 ft. wide, 150 ft. long and 3 ft. deep (see Fig. 5), with cypress gunwales 8 ins. thick at the bottom and 6 ins. at the top, with yellow pine bottoms 2½ ins. thick; with a rake at one end

* By a square is meant 100 sq. ft.

allowing an overlap on the shore for connecting the mat to the revetment above water. The barges were rigidly fastened to each other by beams 8 x 18 ins. in section and 30 ft. long, a 9-ft. space being left between. This space was spanned by 4 x 6-in. sills, at intervals of 5½ ft., which supported the posts, 4 x 6 ins. and 6½ ft. apart, on which the runs rested. The runs or skids were 4 x 6-in. cypress, beveled on the upper side, and were 5½ ft. between centers. The top of the runs on the high side was about 13 ft. above the water, and on the low side they rested in the water about 8 ft. outside of the gunwales. There was a break in the run slope at 37 ft. from the top, the upper grade being about 6½ to 1, and the lower 3½ to 1. The lower ends of the runs rested upon a 4 x 10-in. guard sill, running parallel to the barge, to which they were bolted. A floor 4 ft. below the top of the runs enabled men to work under the mat; it also acted as a brace to the posts. A platform 8 ft. wide projecting from the high side of the ways extended the whole length of the barges. On this platform was a drum on which

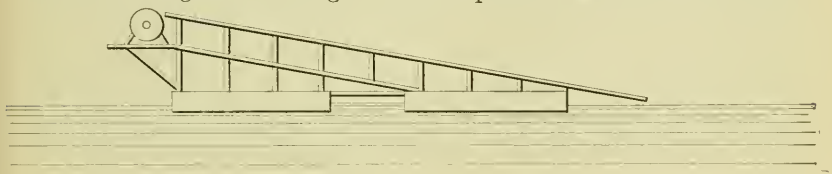


Fig. 5.

was formed the wire netting. It consisted of a wooden frame 12 ft. in circumference, with a 6 x 6-in. wooden shaft. On this circumference were fitted wrought-iron bands 1½ ins. wide and ¼ in. thick. These bands were in pairs, 6 ins. apart, each pair 2 ft. between centers. Around the circumference of the bands were nibs of round iron ½ in. in diameter, 1¼ ins. high, and riveted to the band. The nibs were in pairs also, 1 in. apart, and 12 ins. between centers of pairs. The nibs were exactly opposite each other on the different bands, making continuous lines along the drum.

In making the netting a No. 8 wire was placed along the drum between the nibs, then two transverse wires were placed between the bands, one over and the other under the longitudinal wire. Before the drum revolved, bringing a new set of nibs to the top, a 2-ft. mesh was formed by twisting the cross wires firmly around the longitudinal wire with a steel pin.

In constructing the mattress, longitudinal poles were first laid 8 ft.

apart, not for the purpose of strengthening the mat, but because it was found impossible to launch it without them. On these poles was placed the wire netting of the same dimensions as the proposed mat. On the netting was placed the brush in three layers to the thickness of 1 ft. Then the willows were sewed to the wire netting with No. 12 wire, the seams being made 4 ft. apart, and in lines parallel with the length of the mat. In taking the stitches one man stood on the top of the mat, and another on the platform under it. The lower man carried a coil of wire, playing it out in the line of the proposed seam. The man above carried a $\frac{5}{8}$-in. iron needle, 3 ft. long, having an open slot in one end and a bend in the other. He thrust his needle through the mat at intervals of 2 ft., catching up the wire from beneath in the slot and pulling it to the upper surface, where a wooden toggle was placed in the bight of the loop and fastened to it by a staple, holding it temporarily in place until the entire seam was made, when a top wire was run through each loop, and a turn taken around each loop, thus forming the completed seam. It was proposed at first to build continuous mattresses of this form, but for some cause the purpose was abandoned, and mattresses 50 x 150 x 1 ft. were constructed instead. Cross rows of poles were wired to the top of these mats, to keep the rock ballast from rolling off. The operation of sinking was similar to that employed for the pin and pole mats of 1878.

Permeable floating screen revetments were experimented with again, but the results were not satisfactory, the high water carrying them away. They proved serviceable on the Missouri River, but on the lower Mississippi they were entirely too light in construction to be of permanent value.

Memphis Harbor.—The work for the protection of the river bank at Memphis was in progress at the same time as that at Delta Point, Vicksburg. In 1878, owing to the prevalence of yellow fever, but little was accomplished, but the next year a full and successful test was made of the revetment method of improvement for that locality. Neither the appliances for manufacturing the mat nor its construction were similar to those adopted at Delta Point. Stationary mattress ways were formed by cutting the bank to an angle of 40° and placing the slope posts 5 ft. apart, on which rested 4 x 6-in. stringers rounded on top, and drift-bolted into the posts. In the mat construction various designs were tried. At first four frames of willow poles

were used, the first frame at the bottom being formed by placing poles 4 ft. apart, and other poles transverse to them at the same interval. These poles were fastened together by ¾-in. oak pins. On this framework was placed a layer of willows; then another framework in 12-ft. squares; then a second layer of willows, and another framework in 12-ft. squares; then a third layer of willows, on top of which was placed the last frame of poles 4 ft. apart, as in the bottom frame. Each intersection was firmly pinned through the mat, and fastened also by No. 12 annealed wire, thus forming a very thick mattress. This structure was found to be unnecessarily heavy and thick and was therefore changed, the two 12-ft. pole frames being dispensed with, and only two frames used, one at the bottom and one at the top, with poles 4 ft. apart pinned every 8 ft. by 1-in. oak pins, and well wired. These mats were sunk in water 80 ft. in depth at some points. The sinking was accomplished in a manner similar to that followed at Delta Point. The stone at this locality cost $2 45 per cubic yard on barges at the work. Up to June, 1880, 1 300 lin. ft. of bank had been protected at this point, and in addition to the subaqueous work, upper-bank revetment was constructed of the same design as the mattress, making a total of 2 385 squares of revetment. The distribution of material in this work is shown in Appendix A. Up to 1882, 3 125 lin. ft. of bank had been protected, and the caving was reported at least temporarily stopped.

In 1881 the pin and pole method of mat construction was abandoned here, and a wire frame substituted. No. 10 annealed wires were laid longitudinally and 6 ft. apart on the same ways used for the old mats, the end of each wire being turned about a binding pole and held temporarily by a strong stake. At right angles to these wires and 6 ft. apart others were also fastened at the ends to binding poles and temporary stakes. At the intersections these wires were fastened by other wires of sufficient length to reach well up on stakes to which they were temporarily attached. These stakes were driven into the ground and reached an elevation 2 ft. above the mat, preventing it from sliding off the ways during construction. On the bottom wire frame thus formed, willow brush in four layers was laid to a thickness of 18 ins., and over it a wire frame similar to that below was placed, and fastened to it through the mat by the short wires that were temporarily held up by the stakes. At the ends the wire frames were fastened to a sys-

tem of poles which formed, when joined together, a strong selvage around the mat. These new mats were 125 x 50 ft. and 18 ins. thick. They were launched from the ways and sunk in a manner similar to the others, with the exception that a portion of the ballast was gravel in sacks tied to the mat and loose. This proved more economical at first than stone, but was neither so satisfactory nor so permanent, being too light to resist the force of the current. During the same season, 1881, a part of the upper bank was revetted in the following manner: First, brush was laid on the bare slope in three layers to a thickness of 18 ins.; then wire was placed over it and fastened to specially formed stakes, which were driven into the ground, stretching it tight over the brush, thus binding it down; then the brush was given a good covering of stone. About 900 squares of this mat and 130 squares of upper-bank revetment were constructed, with the expenditure of material shown in Appendix A.

New Orleans Harbor.—The early revetment work at New Orleans was to prevent the wharves from being washed into the river. In 1878 the project consisted of the erection of a bulkhead built of pairs of piles, the piles in a pair being 3 ft. apart, and each pair 6 ft. from the center of the other pair, all connected at the top and low-water line,* and of the sinking of cane mats along the subaqueous slope.

The mattresses were constructed of fishpole cane, being, when completed, 2 ins. thick, 25 ft. long and 24 ft. wide. At first they were made by hand, two galvanized iron wires being used as the warp, woven with the shoemaker's stitch, one wire running over one cane, and under the next, over the third, under the fourth, and so on, the other wire being reversed, thus crossing the first and forming loops, but being fastened only at the end canes. A split was made in each cane through which it was hoped sediment would pass and help to sink the mat. Seven double strands of wire about 4 ft. apart were used for each carpet. Afterward a loom was used, and spun yarn substituted for wire.

When eight of these small mats or carpets were completed, they were taken to the floating ways and placed thereon and connected, making a mattress 200 ft. long by 24 ft. wide. At first they were ballasted with old iron boiler tubes filled with sand and fastened to them. They were ballasted on the floating ways, the ways being after-

* Behind these piles and up to the low-water line were to be piled brush fascines, forming a brush wall.

ward moved up to the line of piling forming the bulkhead, and the mattress fastened to it by iron rings. The ways were then pulled out in the river by a tug, and the mat thus launched was sunk in place on the bottom. Stone was not used, as it was deemed necessary on this steep subaqueous slope to fasten the ballast to the mat to prevent its sliding off. The cost of a square of this cane carpet was $12 87. In 1879, owing to the change from hand weaving to loom work, and the experience gained by the employees during the first season, the cost was reduced to $9 88 per square.

Seventy-four carpets were made, averaging 200 x 26 ft.; 1 116 lin. ft. of bank were revetted, and about 1 496 squares of river bed covered, at a cost, as stated, of $9 88 per square, and $13 38 per lineal foot. The officer in charge reports it was suggested that shrimp, which are known to eat the oakum out of the seams of vessels lying in port, might attack the spun yarn used in construction of the mats, but experiments indicated that there was nothing to be feared from this source.

In 1880 a contract was entered into for the sinking of mattresses at 65 cents per square yard, or about $7 22 per square. Anchorage piles were driven as before, and cane mattresses 200 x 70 ft. sunk with sand bags. The mats were made of 40 sections, 14 x 25 ft. each, sewed together. For mattress construction, a float was used formed of eight coal barges fastened together, supporting launching ways, the dimensions over all being 170 x 206 ft. The skids had a slope of 7½ ft. in 170 ft.

An attempt was made to construct and sink a mattress 140 x 200 ft., but it proved a failure, so the ways were changed for mattresses 84 x 200 ft., and the slope of skids increased to 8 to 1.

For the purpose of mooring the mattress in the desired position preparatory to sinking, three flatboat gunwales were used, so placed as to form a right-angled triangle with the altitude along the piling and the base normal to the shore, the diagonal barge being supposed to fend off drift. To the barge forming the base of this triangular mooring plant, the mattress was fastened, and, after launching, also to the ways. It was thus sunk with guides on three sides, with lines to the gunwales above, to the ways barge below, and to rings on the piling of the bulkhead. After sinking about five of these mattresses, the contractor's plant was destroyed by drift, and the work abandoned. Work was commenced again under a new contract later in the same season, and

27 cane mats 200 x 84 ft. sunk successfully, covering about 2 268 lin. ft. of bank.

In 1881 a board of officers was appointed by the Secretary of War to consider the value of revetment placed up to that time, and the desirability of a change in the project and methods of construction.

After a survey of the locality had been made and the subject of improvement thoroughly studied, the board submitted a report condemning the plan on which the work was based and the methods of construction adopted, as ineffectual in accomplishing the desired end, and recommended that changes be required by the city government in the construction of wharves by companies and individuals, and also that at certain points mattresses be sunk 400 ft. wide, made of brush, not cane poles, costing about $28 per lineal foot. A project based on the report of this board of engineer officers was adopted by the Mississippi River Commission, but no further work was undertaken until the season of 1883.

It may be well to state here, for those not familiar with the Mississippi River and its phenomena, that below Baton Rouge, which is 1 000 miles from St. Louis, or 820 miles from Cairo, its general appearance, physical features and properties and mechanical forces are quite different in degree from those existing above. The slope is less, the average depth much greater, the width less. No obstructive bars or crossings occur in this portion of the river. The caving is comparatively slight. The range from low to high water at New Orleans is about one-third that at Arkansas City. The current velocity at low water is very low, and the radii of the bends are greater.

With such a radical difference in the agencies of bank destruction it can readily be seen that a difference in means for protection will follow. The subaqueous work is placed with greater facility and a much lighter form of revetment is practicable and efficacious. Interrupted or broken revetment and dikes with unprotected intervals produce better results than in the upper river, where the range of water level, currents, depths and instability of the banks are so much greater.

Early Work of the Mississippi River Commission.—In 1879 Congress provided for the appointment of a Mississippi River Commission, composed of three United States engineer officers, an officer of the Coast and Geodetic Survey and three civilians, whose duty should be to obtain

such data as were deemed necessary, on which to base a general project for the improvement of the river from Cairo to the mouth, and execute the same with funds appropriated for the purpose. By 1881 this commission had accumulated a vast amount of information, had made an exhaustive study of the problem, and formulated a project for improvement, which was inaugurated in 1882.

The plan adopted contemplated two distinct classes of work.

First.—Permeable dikes, screens, etc., closing chutes and side channels, thus contracting or narrowing the river in the wide and shallow reaches, and making it conform to a normal low-water section.

Second.—Bank protection work, brush mattresses, and stone riprap to hold the shore or river bank intact, preventing caving and the forming of bars below. To quote the report of the commission to the Secretary of War, dated November 25th, 1881:

"The bank revetments are intended, not only to stop the constant, and in some localities very rapid, enlargement produced by erosion and caving of concave bends, but in addition thereto to check the growth of bars and shoals below, by accretions supplied directly therefrom.

"The process of laying this revetment will vary greatly in different localities, but will commonly, or at least in many cases, consist in first freeing the banks of snags, stumps and brush, and then placing a mattress * * * upon that portion of the slope extending from deep water to a few feet above the water level, and weighting it with sufficient riprap stone to hold it in place.

"The revetment to be afterwards completed by grading the bank above the water level to a proper slope with streams of water under high pressure, after the manner commonly followed in hydraulic mining, and laying thereon a supplementary mattress, overlapping the one previously laid, and extending up to the crest of the bank. * * * Whenever it may be deemed safe to omit the mattress on the upper part of the slope, and place the stone covering directly on the bank, that course will be pursued."

Two reaches of river were at first selected for improvement, viz., Plum Point above Memphis, 40 miles long, and Lake Providence below Greenville, 35 miles long. The work of harbor improvement on the lower river, in progress under the United States engineer officers, was also transferred to this commission.

Radical changes had to be made in the general design of protection work, to make it applicable to long reaches of rapidly caving banks. The harbor mattresses, though of temporary value, could not be said

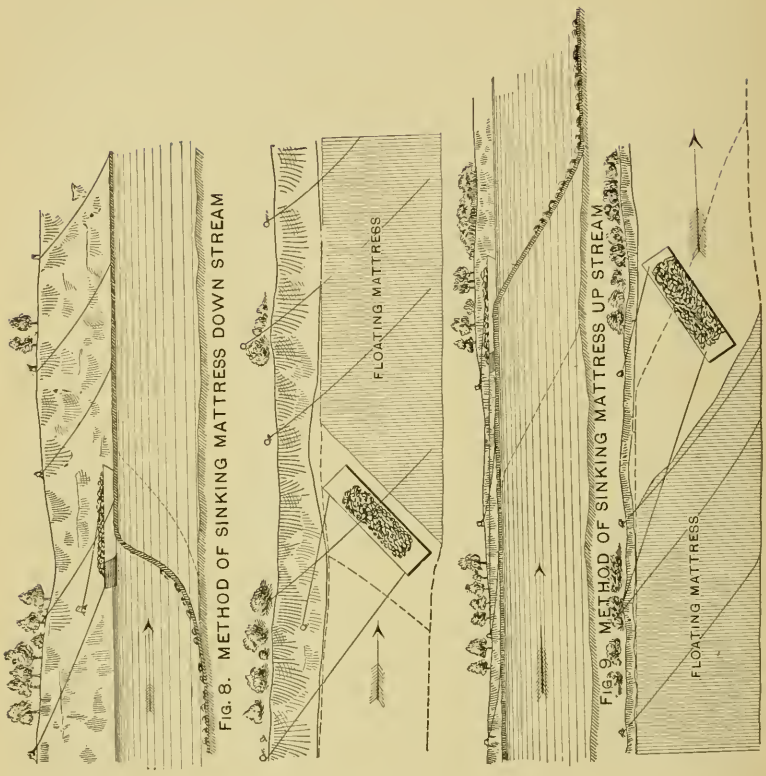


FIG. 8. METHOD OF SINKING MATTRESS DOWN STREAM

FIG. 9. METHOD OF SINKING MATTRESS UP STREAM

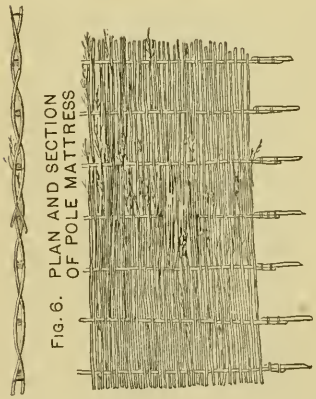


FIG. 6. PLAN AND SECTION OF POLE MATTRESS

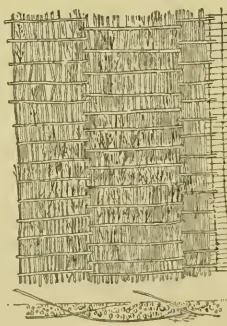


FIG. 7. PLAN AND SECTION OF MATTRESS WITH WIRE BASE

to have given permanent protection, because of their small size and detached condition when in place on the subaqueous slope. For long reaches they were very expensive.

The first considerable change was made by increasing their width to three times that formerly attained, and constructing and sinking them in continuous stretches of from 500 to 2 000 ft. Where timber skirted the bank, much clearing and grubbing had to be performed, and in many places great numbers of snags had to be removed by boats built for that special purpose. The thickness of the revetment was very much decreased and the strength much increased by means of iron rods, cables, etc.

The first mattresses used by the Mississippi River Commission on the Plum Point Reach were, when completed, much the same in general form as those constructed at Delta Point in 1881, though the manner of making them was different. They were constructed on a specially designed machine barge and had for their foundation a wire netting of No. 8 and No. 12 wire, the heavier running longitudinally up and down stream, and the lighter transversely, the distance* between the former being 4 ft. and between the latter generally $2\frac{1}{2}$ ft. The brush was carried by machinery operated by steam, from a brush barge, over the mat barge and deposited on the netting, where it was received by men holding hooks, who packed it close together in continuous layers. After a full shift of brush had been deposited upon the netting, brush binders were inserted with their butts into and at least 8 ft. through the netting (see Fig. 7), the tails being bent over the butts last laid and wired firmly through to the netting. The binders were placed at right angles to the brush, and wired at a point directly under the space to be occupied by the succeeding shift. These mats were continuous, ballasted and sunk in the same manner as those to be described hereafter.

A more detailed description of the wire net making machine and general construction of this mat will not be given here, as it was found to be unsatisfactory for many reasons and was soon abandoned.

Work at Delta Point, 1882–83.—In 1882 little or no bank protection was undertaken at Plum Point or Lake Providence Reach or any of the harbors except Vicksburg, where the revetment of Delta Point was vigorously prosecuted with new methods of construction. The

* See the "Report of the Mississippi River Commission," 1883, p. 371.

new mattresses were built after a plan adopted on the upper river in the vicinity of St. Louis and were termed woven mattresses. These new mats were made on the ways built for the wire-net construction heretofore described, the drums and other wire-net machinery having been removed.

The mode of construction was as follows: A low-gunwale flatboat, acting as a mooring barge, 170 ft. long by 26 ft. wide, was placed with the bow to the shore and stern into the stream, being held in that position by 1½-in. manilla lines leading to deadmen on the bank and anchors in the stream above. On the lower down-stream side of this boat, and firmly secured to it by 1-in. lines, were the ways, placed with the lower ends of the runs up stream and the bow of the barge against the bank, the position of these two boats being directly over the

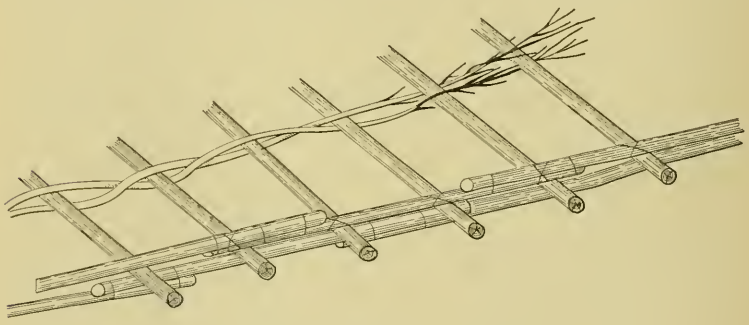


Fig. 10.

upper end of that portion of the lower slope of the bank to be revetted. The construction of the mattresses was commenced by laying 30-ft. willow poles 5 ins. in diameter at the butt across the runs and near their lower ends. These poles were lapped from 4 to 6 ft. and fastened together with No. 12 wire and four or more 6-in. boat spikes. The number of these poles depended on the width of mattress desired.

Longitudinally between the runs, about 6 ft. apart, were placed willow poles of the same dimensions, with their large ends resting on the transverse system and tightly wired and spiked to it as shown in Fig. 10. On the large ends of these longitudinal weaving poles, a third set similar to the first was laid transversely, and all three firmly spiked and wired together, thus forming a mat head or cross-selvage for the woven mat which kept it from unraveling, and acted as a strong fastening in sinking, etc.

Before commencing to weave the mattress this upper frame or head was fastened to the mooring barge by 1-in. lines. Willow brush was then woven over and under the longitudinal poles, the brush being about 30 ft. long and 3 ins. thick at the butt. The weaving was accomplished by placing the brush over one pole and under the next, over the next, and so on, until its end was reached, when it was forced down the runs and poles until the transverse poles or head was reached, to which it was made to fit tightly by means of mauls. This operation was carried on all along the ways by different gangs of men and repeated until the 30 ft. of poles, or one shift, was nearly filled with woven brush, only space enough being left on which to spike and wire other poles, thus making a continuous wooden thread on which to continue the weaving. Every 100 ft. transverse poles similar to those at the head were placed across the mat to give it strength, and longitudinal poles 12 ft. apart were bound to it on top, forming cribs to keep the rock from falling off when sunk on a steep subaqueous slope.

When one shift, or about 30 ft. in length of the mat, was woven, it covered the entire ways, and in order to continue the weaving it became necessary to remove this. To do so the lines holding the ways to the mooring barge were slackened and the ways moved down stream from under the mat, the movement being produced by the force of the current. When the latter was not sufficiently strong, the ways were moved by capstans and lines leading to anchors or the shore below. They were moved just the length of a weaving pole for each shift, until the end of the mattress was reached, when a strong frame selvage similar to the head was constructed, and the last shift launched into the river. The mattress thus floating was held at the upper end by lines to the shore and to the mooring barge, and on the shore side by 1-in. lines leading to deadmen or to trees 30 ft. apart.

These mattresses were about 400 ft. long by 144 ft. wide, the method of sinking being very similar to that employed for the earlier designs.

Rock barges were brought to the outer edge of the mat and fastened to it with lines at intervals of 10 ft. It was then loaded uniformly with rock until under a heavy strain; then, at a signal, these lines were cut, and, as it settled below the surface, the barges were pulled over it and rock thrown on it in quantities sufficient to insure

it remaining on the bottom. The number, size and position of the lines (manilla rope) depended on the force and direction of the current or currents and the depth of water. After each mattress was sunk, sharpened cottonwood piles 40 ft. long and 15 ins. in diameter were driven through the shore side, to keep it from sliding out into deep water where the slope was abrupt. From 60 to 70 ft. of this form of mattress was made per day with 52 men; 30 weaving, 6 at the mauls, 6 binding poles, 4 linesmen, and the rest on the material barges.

The mats were made to lap about 15 ft. when sunk, so that no unprotected seam would be left in the revetment. The piles used were driven about 25 ft. apart and 20 ft. into the ground, being cut off at a stage of about 10 ft. above low-water mark.

After the subaqueous slope had been covered in this manner, the upper bank was graded with shovels as in former years, but this proving very slow and expensive, sluice grading was attempted with fair success.

A trench was first cut with a shovel to the required angle of the slope, 2¼ to 1, and in it was placed a continuous line of wooden boxes, one fitting into the other, forming a trough reaching from top of bank to water surface. A floating pile-driver containing a pump for sinking piles by water-jet was moored near the trough, and supplied its upper end with a stream of water through a hose. The material of the bank was excavated and thrown into the trough with shovels, the stream of water carrying it into the river. After grading as far as the laborers could throw the earth from the shovels, the trough was moved and the slope carried ahead. This method was also abandoned for hydraulic grading on a very much larger scale, the water being forced directly against the bank from a nozzle, to which it was supplied by very powerful pumps built for the special purpose of river-bank grading.

The grader used was designed for the Mississippi River Commission for the special purpose of bank grading. It consisted of a barge 110 ft. long by 30 ft. wide, with 6 ft. depth of hold, well braced and chained to carry the heavy hydraulic machinery. The machinery consisted of a Blake duplex compound condensing pump. The pump had double outside plungers, each 16 ins. in diameter with 24-in. stroke. The high-pressure cylinder was 18 ins. in diameter, and the low-pressure cylinder 36 ins. in diameter, the stroke being the same as

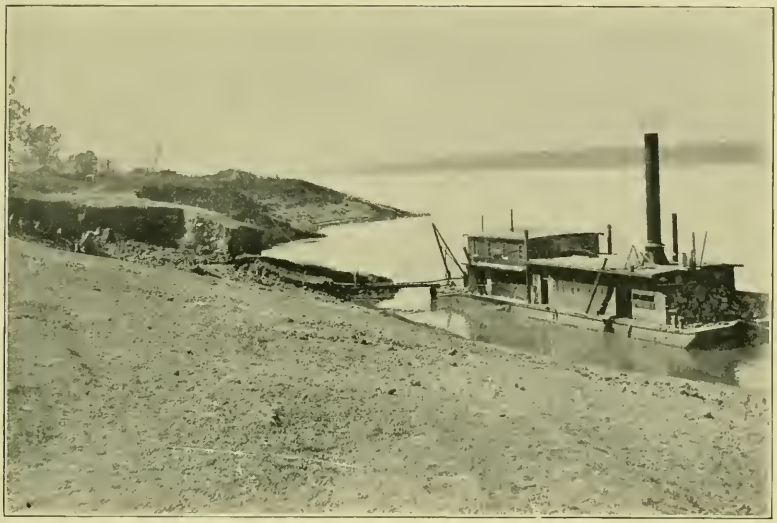


FIG. 1.

FIG. 2.

that of the plungers, 24 ins. The capacity of the pump was 2 000 galls. per minute under a pump pressure of 160 lbs. and a steam pressure of 80 lbs. per square inch.

In connection with the low-pressure cylinder and boiler there was an air pump and condenser, which condensed the steam and heated the injection water. The steam supply was obtained from a battery of three boilers 42 ins. in diameter by 26 ft. long, with five 10-in. flues each. These pumps sucked the water from the river through two 12-in. supply pipes, and forced it through two 12-in. discharge pipes to the forward end of the boat, where they entered a 14-in. boom pipe, which was 65 ft. long and but 10 ins. in diameter at the upper end, and was held in place with accompanying stages by means of shears and steel ropes, the angle made by it with the plane of the deck being governed by a hoisting engine situated in the bow of the boat. The boom pipe had twelve 4-in. openings, with Chapman valves, to which either 4-in. or, by means of reducers, 2½-in. rubber hose was attached. At the end of each hose a 1¾-in. nozzle was used to concentrate the water for the purpose of cutting the bank. The boom pipe and stages were subsequently removed, and the hose connections made directly from the barge (see Plate III, Fig. 1). When all was ready for grading, the bow of this machine was placed against the bank, and the boom pipe and stages lowered. The hose was laid leading from the opening in the pipe to within about 8 ft. of a face which had been cut in the bank as a slope guide, and also as the only practicable way of under-cutting the steep bank and carrying forward the grade. The nozzle, which was attached to the end of the hose, rested on and was turned on a swivel, moving in an oak frame or trestle spiked to the ground. Three nozzles were used, each worked by three men. The slope was cut a little ahead at the upper end, the reason being that by so doing the water, after spending its force on the bank, on seeking a lower level ran close to the lower edge of the face, helping to undercut it.

The direction of the stream was always kept at a greater or less angle than 90° with the face, as otherwise it would form a pocket in the bank and a water cushion to resist the pressure. It was found that a 2½-in. hose did not work advantageously, owing to the frictional resistance. A 4-in. hose and 1¼-in. nozzle gave the best results. The bank was graded to a slope of 2¼ to 1. Sand and light deposit was

carried off rapidly by the force of the water, but the tougher or more tenacious materials, such as clay and buckshot, of which the Delta Point bank is in great part composed, resisted the jet for some time, but eventually was reduced. With three nozzles an average of 1 300 cu. yds. was removed per day, at a cost of about 4 cents per yard. The inequalities left in the slope were dressed down with shovels or sluice.

After the bank was made comparatively smooth and regular, continuous lines of poles, 30 ft. long by 5 ins. in diameter, well wired and spiked together, were laid on it parallel with the top, the rows being 10 to 12 ft. apart. On these were transverse poles, spiked and wired together in the same manner, in rows reaching from the top to the bottom of the slope, also 10 to 12 ft. apart. These rows were wired together at the intersections. Upon this frame, willows, 20 to 30 ft. long and 2 to 3 ins. at the large end, were laid with the butts up hill, the bushy or leafy part of the top being covered by the butts of those below, each succeeding layer of brush having its butts over the tops of the preceding one, until the water edge was reached. On the brush were placed lines of longitudinal poles directly over those in the bottom frame, and fastened to them by No. 8 wire. This revetment at the water's edge was fastened to or overlapped the mattress work if possible; otherwise a connecting mat was built to make the protection continuous from near the top of the bank to the river-bed. On the upper-bank revetment rock was placed in just sufficient quantity to cover the willows.

The cost of the Delta Point work just described during the season 1882–83 is given in Appendix A. The cost of brush obtained by Government labor was $2 53 per cord; poles, 32 cents each; rock, by contract, $1 90 per yard; wire and spikes, 5 cents, and nails 4 cents per pound.

The total cost of protection, not including grading, was:

Mattress work.....	$2 90 per square.	$4 17 per lineal foot.
Upper bank revetment..	6 89 " "	5 17 " " "
Total.....		$9 34 " " "

Including towing, grading and other expenses, the cost per lineal foot was about $14.

Work Done in 1883.—In 1883 revetment work was carried on by the Mississippi River Commission on the two principal reaches, Plum Point and Lake Providence, at Hopefield Bend, opposite Memphis, at Memphis, Delta Point and New Orleans. Practically the same methods were employed in 1883 at all points above the mouth of Red River as were used at Delta Point the year previous.

At Bullerton 100-ft. mattresses woven on a wire base were used, the willows being interlaced, as shown in Fig. 7. At Plum Point hydraulic graders were worked similar to that described. The local engineer reported 1 800 to 4 000 cu. yds. of material moved per day at a cost of 2.98 cents per yard.

Electric lights were used for night work. Mattress boats of improved make, over 200 ft. long, but practically the same design as those described, were used. Barges were employed specially adapted to the work. In fact, an entire floating plant had been constructed or purchased specially designed for revetment construction, which greatly reduced the expense and added to the strength and efficiency of the finished protection work. It had been the custom in the work previously performed to press into service anything in the form of a barge that could be used and obtained at small expense, making temporary changes in construction, rather than expend the inadequate appropriations in procuring floating property of special design.

In 1883 wire shore fastenings, instead of manilla ropes, were introduced on a small scale. Fig. 6 shows the woven mats used at Plum Point, and Figs. 8 and 9 the two methods of sinking them when continuous, one with and the other against the current. The revetment was similar to that described. The grading at Bullerton cost from 3 cents to 3.8 cents per yard, the force employed being one nozzleman and two helpers for each nozzle. These men were furnished rubber clothes by the Government.

The cost of the bank protection work on the Plum Point Reach during 1883 was as follows: Mattress work, $6 30 per square; upper-bank revetment, $6 40 per square.

At Memphis the small mattresses were still in use, constructed on shore ways and towed to the required sinking locality. They were not made of quite the same thickness. Bank grading was still executed with shovels. The slope and revetment were thoroughly drained above low water by connecting the city drains with special wooden

culverts. The upper-bank protection was slightly different from that used in former years. Wire was laid on the slope forming a 6-ft. mesh. On this was placed 6 ins. of willow brush and on top of the brush wire similar to that below, the top and bottom wire frames being sewed through the brush by separate wire, thus forming one continuous mat. This revetment was secured by placing on it stone, sand in bags and earth. The distribution of material in this work was as shown in Appendix A.

At Hopefield Bend the mattresses were of the woven type, from 250 to 300 ft. long by 140 ft. wide, made in a similar manner to those of 1882 at Delta Point. The upper end before sinking was fastened to the mooring barge by ten 1-in. lines. Two 1½-in. or 2-in. lines were attached to the mattress by means of pin and shackle, and led to deadmen or trees on shore, both passing under the mooring barge. In sinking, the mattress was first ballasted by placing running plank on it and having the laborers carry and distribute rock evenly over it, thus destroying its flotation. When the weight commenced to strain the small lines fastening it to the mooring barge, they were slackened slowly and the mattress allowed to sink until the upper end rested on the bottom. Then a barge loaded with rock was pulled over the mat, being directed by lines from the mooring barge and bank, and the rock was thrown out, sinking the mat. At Delta Point, the fastenings between the mooring barge and mat were cut, the latter sinking rapidly.

The revetment above the low-water line was the same as at Delta Point. Some trouble was experienced from floating drift which accumulated in front of the mooring barge and under the head of the floating mattress, making it difficult to sink the latter. Many expedients were tried to prevent the destruction or loss of work through the pressure of this drift. Diagonal booms and chains were used, but without a great amount of success.

At Lake Providence Reach during the season of 1882 and 1883, the following modifications were made in the bank protection work. In the woven mats alongside of every weaving pole or every alternate one, depending on the strength required, iron rods with welded eyes at each end were placed as the mat was built, the ends of the rods being connected by lap rings or clevises. These rods added longitudinal strength to the mattress, which was much needed in strong currents

and deep water. In addition to these iron rods, at certain localities where great strength was essential, twisted wire cables were placed on the mattress longitudinally, secured at intervals to the weaving poles by short wires. These mattresses were loaded at first next the shore, and then the head was heavily weighted so it would go to the bottom quickly.

The cost of grading varied from $3\frac{1}{3}$ to $2\frac{2}{3}$ cents per cubic yard, but the material excavated was sand, which was moved with great facility.

The engineer in charge estimated, after making daily observations for over a month, that to excavate 1 cu. yd. of earth took a fraction less than 1 cu. yd. of water under a pressure of 140 lbs, with a steam pressure of 80 lbs. and a vacuum of $26\frac{1}{2}$ ins. With a steam pressure of 80 lbs. it took 3 lbs. of coal per cubic yard of water thrown or earth excavated. He also estimated shovel grading at 30 cents per cubic yard, or ten times the cost of the hydraulic work. No change was made in the upper-bank work.

The cost of the protection work at Lake Providence Reach during 1882–1883 was $11 20 per lineal foot.

At Pilcher's Point a change was made in the manner of handling the mattress by the introduction of what was called a detachable mattress head, which was used in place of a mooring barge. Figs. 11, 12 and 13 show the head and details of construction. It was made of wood and consisted of two chords 20 ft. apart, connected by a series of cross diagonal bracing and a hog chain as shown in the drawings.

"The up-stream chord was 12 ins. wide by 30 ins. deep; the lower one 12 ins. wide by 19 ins. deep. These chords were 163 ft. long and had a camber of 6 ins., and were constructed of pine planks."

During the construction the mat was fastened to this head by lines which were sunk with it, being released by means of a lever and trips.

One mattress sunk was 152 ft. wide and 1 228 ft. long. The method of using the head was as follows: The sinking was commenced near the center, the mattress being loaded with rock both ways from this point until it had reached a depth of from 10 to 30 ft. Then two rock barges were lashed together and placed about 50 ft. from the head and allowed to drift over the partly sunken mattress, rock being thrown on the latter until it reached the bottom. Previous to the final sinking the depth of water over the outer edge of the mattress head was taken by means of a sounding line fastened to it. When the mattress had

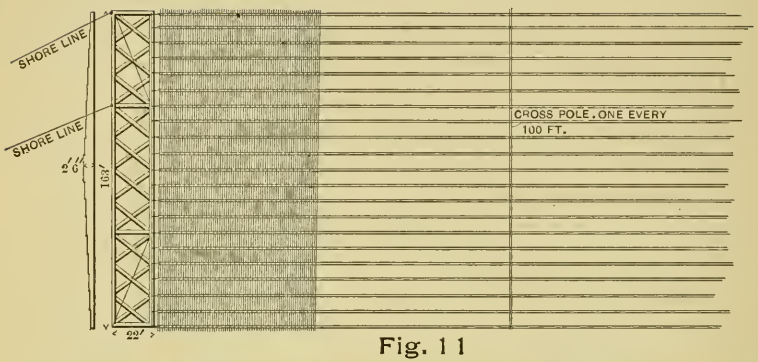


Fig. 11

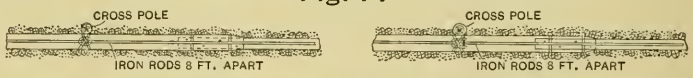


Fig. 12

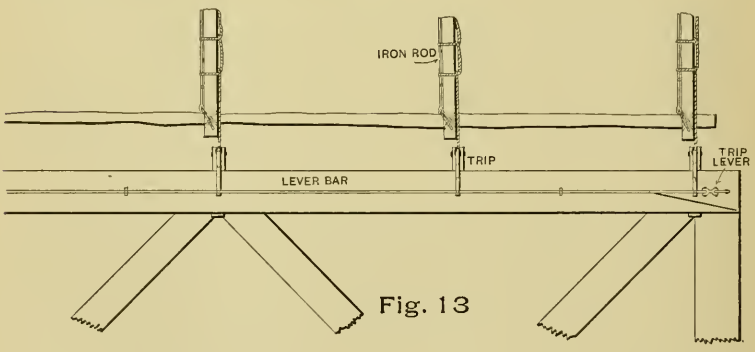


Fig. 13

sunk to two-thirds the entire depth, ten men, by pulling a line made fast to a ring at the end of a lever, tripped the triggers, releasing the head at once; the mattress sank with its load of rock to the bottom, and the head rose by means of its buoyancy to the surface. This mattress head was not generally adopted, the old method of mooring barges being adhered to, even to the present time.

The first work of revetment construction under the direction of the Mississippi River Commission at New Orleans was commenced in 1883.

Floating ways 400 ft. long were constructed by linking ten barges together, each 100 ft. long by 20 ft. wide. The woven mattress was used, the weaving poles having iron rods fastened to them. The poles and rods were 25 ft. long, the rods linked together at the ends making a continuous weaver. Two No. 8 wires were run through and across this mattress every 12 ft., and also heavy binding poles transversely every 25 ft.

After a very high stage of the river in the season of 1884, it was found that in many places in the long reaches of protected bank, the revetment both above and below the low-water line was too light to prevent the caving, and at some points the work was considerably damaged, necessitating changes in methods, strengthening and enlarging. Much unfinished work was destroyed because of lack of funds.

Work at Memphis in 1884.—One of the localities to suffer during the flood was the Memphis front. There the channel directly against the bank deepened materially, undermining the small mats sunk in former years. To prevent further damage and in order to replace the destroyed work, mattresses varying from 150 to 300 ft. in width were sunk and the upper bank graded and paved with stone.

The 300-ft. mat was constructed by placing two coal shells together end to end as a mooring barge, and two mattress barges also end to end on which to weave the mat.

In the construction of this 300-ft. mat two $\frac{5}{8}$-in. iron rods, ten $\frac{1}{2}$-in. iron rods and three $\frac{5}{8}$-in. wire ropes were used longitudinally. It was secured to the mooring barges by 1-in. and $1\frac{1}{4}$-in. manilla lines every 16 ft. across the head. Seven shackle lines from $1\frac{1}{2}$ to 2 ins. in diameter ran from the head to fastenings on the shore, and the mooring barges were held in place by nine lines $1\frac{1}{2}$ to 2 ins. in diameter

leading also to the shore above. Lines were placed diagonally across the mat every 100 ft. and fastened to the shore. This was one of the first very wide mattresses constructed, and notwithstanding its additional strength and the increase in fastenings, it was broken up in sinking. The engineer in charge of this work does not report the cost of this large mattress, but estimates for upper-bank revetment 100 ft. wide, $6 80 per square, and for all protection work, upper slope 3 to 1, bank work 100 ft. wide, and subaqueous work 150 ft. wide, $17 per lineal foot. During this same season on the Plum Point Reach the width of mattress was increased to 175 ft., one mattress being constructed of this width and 2 010 ft. long, another 175 x 1 750 ft., and a third 175 x 1 713 ft. Five wire cables were used in the body of these mats longitudinally.

The grading for shore protection was done with hydraulic power at 2 cents per cubic yard.

The distribution of material was as shown in Appendix A.

There were minor differences made in form and mode of construction of the revetment at certain localities to conform to special conditions and emergencies, which will not be mentioned.

The more important changes made in the old method of construction at Hopefield Bend opposite Memphis, in 1884, were the increase of mattress width to 150 ft. and the driving of a line of piles 8 ft. apart at about 5 ft. below low water, against which the shore edge of the mattress rested before it sank, not allowing it to slide up the bank on a rising water.

The mattresses were made stronger and thicker by weaving brush around but two poles, and then laying the tails over instead of weaving to the end, as shown in Plate III, Fig. 2. This allowed of much closer weaving, the tails being secured by binding poles wired down to the weavers, to which also transverse poles 8 ft. apart were fastened. Five ½-in. iron rods and two ⅝-in. wire ropes were run through the mattress longitudinally, and every 40 ft. transverse wire ropes or ½-in. iron rods were used, fastened to the piles at the shore end. On the upper-bank work wire rope was also introduced and much more stone ballast was used. The shore or upper-bank revetment was extended outside of the line of piles, lapping the mattress about 20 ft.

Spur Dams at New Orleans, 1884.—At New Orleans Harbor in 1884, a continuous mattress 400 ft. wide was broken up after sinking, and as

it proved a very difficult matter to place that type of revetment along the uneven line of wharves, some intact and some wrecked, the plan was abandoned and a new method adopted, viz., the placing of submerged spurs normal to the bank at intervals of 500 to 1 600 ft., generally at salient points. These spurs consisted of a woven mattress foundation 200 ft. wide, of the ordinary form then in use, with rods and cables to strengthen it, extending out into the stream about 350 ft. from the low-water contour, with a narrow cribwork filled with rock on the edge around three sides. When in place and ready to sink, this mattress was fastened to barges on three sides by double lowering lines 1-in. in diameter 16 ft. apart. Each lowering line passed around a timber head on the barge and led thence through the mat and up to a main rope which was carried to a capstan near the shore. All these lowering lines were fastened to the main line, which was manipulated by one man at the capstan.

The shore edge of the mattress was connected by iron rods with deadmen on the bank. Six large toggle lines were fastened to the upper end of the mattress, running thence to shore connections above. The method of sinking was similar to that before employed.

On this mattress on a line parallel with its up-stream edge and 70 ft. below it were sunk in the same manner six wooden cribs about $5\frac{2}{3}$ ft. thick, containing pockets in which rock was placed to sink them.* The bottom crib was 60 ft. wide, and the top one 22 ft. The completed cribwork was about 300 ft. long. The bottom crib was made the length required to fit the contour of the bank, the others varying in length in order to give the completed dike a top surface slope of about 3 to 1 from the low-water mark to the river-bed.

The cribs were made of sawed timber frames connected by long iron bolts and wooden posts fastened with wooden pins, between which and forming the body of the crib was placed willow brush, pockets being left in the construction in which to place the rock for sinking. In sinking the mattress about 7 lbs. of rock was used on each square foot of surface, the crib taking about 7 lbs. for each cubic foot of structure.

The mattress cost $7 60 per square, and the crib $3\frac{1}{3}$ cents per cubic foot. The total cost of one completed spur was about $12 525.

* The number of cribs in a dike varied with the depth of water, cross-section of banks, etc.

Results of Early Works.—By 1885 experience had demonstrated very forcibly the necessity of strengthening all the work of revetment or bank protection that had to withstand the continued force of the current and prevent the sloughing of the bank. Two very active forces to contend with were the seep water that undermined the slope, and the drift that accumulated at the head of the subaqueous work during construction, tearing out the fastenings by the increased pressure and doing great damage to the mat proper. The old mattresses were small and easily handled during construction, but when in place proved but a temporary protection, except at points where the material composing the bank was very hard and tenacious and the current and depth comparatively slight; whereas the new work offered great resistance to the current, especially while sinking, necessitating many more and very strong fastenings. This fact was not thoroughly appreciated until many accidents had occurred. Even during comparatively low water the drift falling into the river from caving banks gives trouble, and when a considerable rise finds a revetment party with an unfinished mattress in an exposed position, the danger from that source is very great. The drift accumulates under the mat head, forming a much greater surface of resistance to the current which has to be borne by the head lines, making the task of getting it to the bottom safely a very difficult one to perform.

At Memphis Harbor in 1885, the head of the mattress was strengthened by more and better iron chains, and by weaving through the mattress near its outer edge one 1½-in. and two 2-in. manilla lines. For a 600-ft. mat 250 ft. wide, sixteen head lines and three diagonal lines were used, nearly all being 2 ins. in diameter. At this time the engineers at all points began to advocate a greater strength of mattress and fastenings.

The methods adopted to prevent the sloughing of the upper bank and general destruction by seep water varied at different localities. When the water came through the bank in well-defined streams, tiles or drains of wood or stone were used. At one place a trench 90 ft. long by 4 ft. wide at the bottom and 16 ft. deep was dug and filled 6 ft. with rock, proving of great benefit.

The distribution of material in 1885 and 1886 is given in Appendix A.

By 1887 the general cost of finished standard bank protection was from about $25 to $30 per lineal foot, depending on the locality,

width of mattress, etc. This included all expenses of repairs and administration.

In this year on the Plum Point Reach the experiment was made of protecting the bank in a certain bend by detached pieces of revetment at intervals of about 500 ft. The results obtained and the discussion of this method will be given further on.

Work Done in 1887–88.—"Prior to 1873 the portion of the river bank at Memphis constituting the harbor front was comparatively stable. In 1876 it had receded 350 ft., and continued to cave at the rate of 100 ft. per annum." The next year, 1877, Congress directed that a survey be made and a report submitted. The plan adopted was for protection by revetting the bank with willows ballasted with rock. This work was carried on without much change in bank line up to 1884, when the new and more substantial revetment was adopted. About that year a complication of changes took place in the river in that vicinity. Hopefield Bend Point was caving away rapidly, thus throwing the resultant of greatest pressure lower on the Memphis side, relieving much of the bank already revetted and threatening the paved levee and the landings below. By 1887 the greater part of this front had been protected, but the continued recession of the Arkansas side, and the accumulated energy in the water passing along the lower revetment on the city front, augmented the caving still further down the river, where the bank conditions are very different from those existing above. Instead of a sloping paved levee from low water to high, the bluff rises nearly vertically to the height of about 100 ft. Here spur dikes somewhat similar to those used at New Orleans were substituted for the continuous revetment. The spurs were placed about 500 ft. apart between centers and reached from the foot of the bluff to the bed of the river.

These spurs were constructed partly by funds contributed by the city of Memphis and partly by Government appropriation. The mode of construction in its general features was similar to that in use at New Orleans. The foundation mattress and cribs, one placed on another, reaching the required heights and inclination, were sunk in practically the same manner. A more minute description of the method of spur construction is given under the head of "Details of Greenville Dike Construction," where the author was in local charge and the work was practically the same as that at Memphis.

In 1888 the whole system of revetment at Memphis comprised a

bank protection of willows and stone about 2 miles in length, and it was estimated that 1 mile more would be required to complete the work. In Hopefield Bend, opposite and above Memphis, it was estimated also that 1 mile farther down stream would have to be protected to hold Hopefield Point, in order to control and guide the channel of the river in the desired direction.

The distribution of material and cost of work during the season at Hopefield Bend and Memphis Harbor are shown in Appendix A. The cost of the Hopefield Bend work from September, 1887, to January, 1888, subaqueous mats and upper-bank revetment combined, including administration, repairs to plant, towing, etc., was $4 44 per square covered.

At Greenville, Miss., a similar project to that undertaken at Memphis was adopted. The dikes were constructed partly by funds contributed by the town. Ten dikes were built, nine of about the same form as those used at Memphis, and one having cribs formed with frames of 3 x 4-in. sawed gum lumber, in lengths of 8 and 16 ft., fastened by ½-in. bolts at each intersection. Here in constructing the upper-bank revetment the old method of carrying and distributing rock on the willows by hand was abandoned, and a carrier devised to take it from the barge and distribute it along the slope. It consisted of a pile-driver placed outside of the stone barge, and a wire rope 1⅛ ins. in diameter, fastened to its leads and passing thence to and over a trestle about 16 ft. high on top of the bank, anchored to a deadman or stump 40 ft. back. A box was suspended from a traveler which ran on the wire rope and was actuated by the hoisting machinery on the driver, and unloaded by a trip line manipulated from the ground.

On this work, the upper portion of the upper-bank revetment had two layers of brush instead of one as had been the custom.

The ten dikes constructed during the season cost, including towing, $101 011 03, or about $10 101 10 each. About 5 000 ft. of bank was thus protected, at a cost of about $20 20 per foot.

At New Orleans Harbor the methods of dike construction were modified in order to get a more substantial form of protection. The iron rods in the cribs were dispensed with and replaced by wooden stanchions, the thickness of the cribs was increased to 6 ft., the depth of rock pockets was reduced to about one-third the thickness; the cribs were loaded as heavily with rock before being lowered to the

bottom as the lowering lines would stand, and no rock was thrown on them after being placed on the mattress. The cost of the New Orleans cribwork was 4 57 cents per cubic foot, excluding care of plant, etc. One mattress cost $7 54 a square, and another cost $9 11. The rock for this work cost $4 11 per cubic yard.

Work Done in 1888–90.—Up to the year 1888 the purpose for which bank revetment had been used was, as has been demonstrated, the protection of valuable property, wharves, landings, warehouses, etc., and maintaining the river in certain reaches within given limits of curvature and position with reference to shoals and bends above and below, in order to regulate low-water depths, etc.

In 1888 it became apparent that unless some action was taken to prevent the caving in some of the lower bends, cut-offs would occur, producing very radical changes in the regimen of the river for many miles above and below. To prevent this occurring at Ashbrook Neck, the upper bend of the peninsula was revetted with the standard type of continuous mattress and upper-bank work. Also at Lake Bolivar, where the bank was receding gradually toward the large inland lake, and the river threatening to turn a portion of its water into the back country, the formation of the outlet was prevented by the use of continuous mattress protection.

This year the mattress work at Plum Point Reach was 200 ft. wide, and was held out to near the low-water line by piling; the upper bank was graded to a more gentle slope. On a portion of the work the brush was not placed on the upper bank, and in its place from about the 13-ft. stage, where the connecting mat ended, to near the top of the slope, stone paving was laid on the bare ground to a thickness of 10 ins. No cost is given of this first paving.

At Hopefield Bend Point and Memphis, the river continued to creep to the west and south, destroying the former point and training the thalweg toward the bluffs. It had been the purpose of the engineer in charge to allow this action to continue until the channel was in the required direction, then to check the caving on the Arkansas side with strong continuous revetment, which, when accomplished, would maintain the best depths and velocity of water along the Memphis city landing. When the time came for putting a stop to the caving at Hopefield Point, funds were not available, and the project could not be fulfilled. The point caved nearly half a mile too far before means

were obtained to check it. By that time a bar had formed at the upper end of Memphis Harbor which threatened to obliterate completely a large portion of the city landing. It is justly claimed that this filling in the upper end of the harbor is not due to imperfect methods or appliances nor to faulty engineering projects, but simply to not having the money to carry out the plans as formulated.

In 1888, 4 105 ft. of continuous standard revetment was placed in Hopefield Bend, a total width of 335 ft., the subaqueous mat being 196 ft. wide, the grade of the upper bank 3 to 1, and the upper-bank revetment 139 ft. wide.

The entire revetment, including mat and bank work, administration, care of plant, repairs, towing, etc., embracing an area 4 650 ft. long by 338.6 ft. wide, covering 15 749 squares, cost $4 75 per square.

At Bolivar Landing in 1888 the mattresses and upper-bank revetments were similar to those used at other points. The former varied in length from 224 ft. to 1 100 ft., and in width from 180 ft. to 250 ft. The cost was $91 341 78, or $6 95 per square, and $21 24 per lineal foot. Including all charges against this work the cost was $9 26 per square, or $28 31 per lineal foot. This latter would be reduced somewhat by allowing for rock unloaded on the bank, but not used, and the towing charges against it.

At Greenville Harbor the dikes, as before stated, were 500 ft. apart, leaving between them an unprotected space of about 300 ft. The foundation mats were about 200 ft. wide, extending out into the river from 375 to 415 ft. At first the dikes caused the deep water to move out toward the center of the river, the depth being increased in the channel. The 30-ft. contour moved out 225 ft. and some deposit showed on the cribs. At the upper end of the system considerable caving had taken place. To check it 706 ft. of continuous revetment was placed 250 ft. wide; also for 700 ft. below the mattress-work the bank was graded, and 250 ft. of it covered with brush. In the interval between some of the dikes sloughing had occurred which had to be repaired by mattress work.

Owing to the tenacity of the soil and the strong work placed at Delta Point, Vicksburg—about 10 000 lin. ft. in 1882-1883—but little repair had been necessary up to 1889, when a small portion of the upper bank was graded back and a revetment placed on it, with diagonal layers of brush, well covered with rock.

At New Orleans the old work placed prior to 1884 was practically intact in 1889. After 1884 the spurs had moved the deep water away from the bank 100 ft., with no increase of maximum depth, as at Greenville. The depth of water at one point where it was proposed to continue the construction of the system of dikes was found to be 150 ft. In order to make the spur conform to this depth and have the required surface elevation, it was necessary to build the cribs 450 ft. long and the mattress longer. The ways not being of sufficient length, a supplementary mat was constructed and placed at the outer edge of the woven mattress, being over 100 ft. wide. This supplementary mattress was made on the plan of those used in the sill dams at the head of the Atchafalaya, mouth of the Red River, and somewhat similar to the old pin and frame construction described in the beginning of this paper. It was made of 2 x 4-in. scantling frames, placed 8 ft. apart. Top and bottom frames, with brush between, were joined together by 2-ft. stanchions, fastened in sill and cap by a 6-in. wire nail and a ⅞-in. pine treenail. The willows on the bottom layer were nailed to the frame to strengthen the mat and keep it from pulling apart. This mattress was sunk 300 ft. from the shore in about 153 ft. of water and in a current of about 5 ft. per second. An additional lowering barge had to be placed outside of those used for the main mat; guide chains from anchors formed of loaded kegs were used also to insure accurate sinking. The mat was loaded and sunk in the customary manner. It cost $9 56 per square; the woven mat, $6 39 per square. The cribwork placed on these mattresses cost 4 cents per cubic foot. This dike when finished was the largest submerged spur ever constructed on the Mississippi River, and probably in this country.

In the autumn of 1889 the work of protecting the bank at Hickman Harbor, Ky., began. Hydraulic grading and the usual method of bank revetment were used. The first mattress was sunk on a projecting point, necessitating the use of anchors in order to get a correct lead for the head lines. The Chinese anchors used, the positions of the barges, etc., are shown in Figs. 14, 15, 16 and 17. The mattresses were 300 ft. wide by 750 ft. long, connected by shore mattresses to the upper-bank revetment. The cost of the work complete was $24 77 per lineal foot.

At Plum Point in this same season all the connecting mats were dispensed with at some points, the mat being held well into the bank,

and a shore revetment constructed connecting with it before it was sunk.

At Columbus Harbor, Ky., this season, crib dikes were constructed, consisting of the ordinary woven mattress carpet, and a single crib, 28 ft. by 8 ft., extending from the upper edge of the mattress to a point 240 ft. beyond the low-water line. The longitudinal axis of the crib was about 12 ft. above that of the mattress.

The cost of the work was 4 cents per cubic foot, $6 70 per square, and $22 65 per lineal foot.

During the season the improvement of Helena Harbor was also undertaken. The project contemplated the placing of 600 ft. of continuous

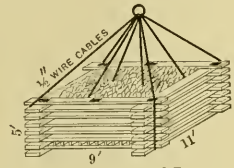


FIG. 14. CHINESE ANCHOR

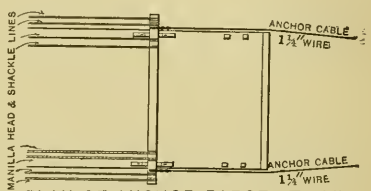


FIG 15. PLAN OF ANCHOR BARGE AT "L"

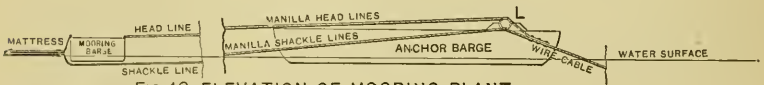


FIG. 16. ELEVATION OF MOORING PLANT

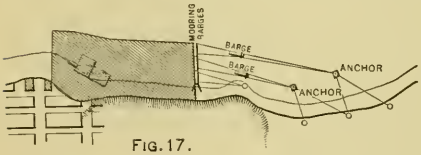
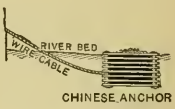


FIG. 17.



revetment and five spur dikes. The mattress work was 228 ft. wide and 613 ft. long. The dike mats were about 200 ft. wide and 330 ft. long, the cribs varying with the slope and depth of the water. The cost of the work was $3 90 per square of subaqueous mattress made, and $4 89 per square of mattress covered.

Seven cribs, containing 355 672 cu. ft., cost 4.4 cents per cubic foot.

In 1889 steel cables were used for the first time, with shackles or toggles as head lines, in place of manilla ropes. Their use had been advocated, but the difficulty of handling them caused engineers to doubt their value from an economical standpoint. Their great superiority over

the manilla line was not so much in their strength as in their durability and the fact that, as compared with manilla, they practically remained of uniform length under the strains produced in mattress sinking. The manilla ropes required a constant taking up and adjusting in order to maintain an equal pressure, whereas the steel cables could be placed at once, and the strains readily distributed throughout the system of head lines, necessitating but little change or further work. This introduction of steel cables as head lines was a very important improvement in mattress construction, reducing to a minimum the chances of loss where a sufficient number of strong shore fastenings were used. A new manilla line when subjected to great strain will stretch far beyond the limit of one that has obtained a permanent set by continued use, and when used among older head lines it produces a lack of unison in the system which is often followed by disaster.

At first the mooring barge cables were fastened to wooden cavels, but it soon became apparent that these would not have sufficient strength, so they were made of cast iron. The shore fastenings were generally deadmen, with keys or timbers 18 to 24 ins. in diameter. At this time it also became apparent that much of the loss by sloughing down of the bank was due to the fact that it was not graded to a light enough slope. Of course, in many places to overcome entirely the tendency to slough, it would be necessary to grade to an exceedingly gentle slope, the angle of repose of wet earth, but this was not advocated, it being deemed advisable to grade to about 4 or 5 to 1 only, thus reducing to a practical minimum the sloughing tendency. The additional cost of flattening the slope was compensated for by leaving from 8 to 10 ft. at the top of the bank vertical, or with its natural inclination prior to grading. It was found that little or no caving took place at the crest when the rest of the bank was protected.

It was suggested that planting Bermuda grass on this upper slope would be advantageous, but this was found to be a poor expedient as a protracted high-water season would kill it. Willow plantations have been more successful, but they add very little strength and cause but slight deposit in the rapid current along the protected bank.

On the Lake Providence Reach, Louisiana Bend, the upper bank was first graded to a slope of about 4 to 1 and then paved with rock to a thickness of 10 ins. The rock was carefully laid by hand directly on

the bank, no willow revetment being placed under it as a foundation, as had been formerly the custom.

Where the upper bank had been previously revetted with willow brush ballasted with stone it was found that in about four years the willows above the low-water line rotted out, necessitating an entire renewal, which meant the removal of the work and redressing of the bank. It was expected that by placing the rock directly on the bank this would be avoided, and by making it thick enough it would prevent abrasion. Where the willow revetment is constantly under water it does not deteriorate except from the mechanical action of running water; whereas, if it be exposed to the air and sun it disintegrates in from three to four years. By thus dispensing with the upper-bank willow work, the first cost of revetment was considerably increased, but it was hoped that its greater permanency would more than compensate for the extra expense incurred.

The mattresses on this work were from 300 to 342 ft. wide and sunk in lengths of from about 500 to 1 000 ft.; they were built and sunk as on the upper river, against a line of piling.

Shore mattresses lapping the main mattress were used when necessary. The main mattresses were strengthened by weaving $\frac{7}{8}$-in. wire cable longitudinally through them, spaced 16 ft. apart, and placing holding-in cables every 16 ft. across them. The latter were fastened to deadmen on the shore, and tightened by tackle blocks after the upper slope was graded.

The grading here was done after the subaqueous mattress was sunk. The engineer in local charge, Arthur Hider, M. Am. Soc. C. E., claimed this to be bad practice, giving among others the three following reasons for grading the bank before building and sinking the mattresses.

First.—The graded bank allowed the holding-in cables to be placed permanently, with the proper strain and position to insure the mattress sinking in the desired place.

Second.—By dispensing with the piling the mattress would sink on a uniform slope, not being canted up on steep uneven ground.

Third.—The rock could be unloaded directly on the finished slope, not necessitating double handling.

The labor cost of the work was $71 679 16 for 27 367 squares, or $2 62 per square.

At Greenville in 1890 the spur dikes were intact, but the current

was attacking the bank at the upper end of the system, threatening its destruction. The dikes could not be considered entirely successful. At some points the bank in the unprotected intervals between the cribs had caved badly. At some the 30-ft. contour had moved out, but at others the reverse was the case, while in much of the improved reach the outer ends of the dikes had been undermined. The amount of deposit between the dikes varied at different seasons, with the change in the stage of the water and its height.

In 1889 the author added two dikes at the head of the Greenville system, and as they were built in a practically similar manner to those sunk in previous years and those adopted for other harbors, a description of the details of construction will cover all the partly submerged spur types in use on the river.

Details of Greenville Dike Construction.—The dikes consist of cribs made of willow poles and brush fastened by wire and spikes, having pockets formed in them in which rock is placed to sink them, and having for a foundation a mattress woven and sunk in the manner heretofore described. They are placed normal to the bank line and 500 ft. apart, and reach out beyond the deepest water in the channel next the shore. The purpose they are expected to serve is the protection of the bank immediately at the point they occupy and the indirect protection of the intermediate spaces by deposit, due to the eddy and dead water formed by their position relative to the swift current of the river.

After an accurate hydrographical survey of the locality to be improved, a mattress 290 ft. long by 290 ft. wide extending about 50 ft. beyond the deepest water was constructed on floating ways, of willow brush and poles, wire and spikes, and launched into the river as woven. After being fastened by cable shackle lines to the shore above, it was ballasted and then sunk in the usual manner.

The mattress was the foundation for the main body of the dike, which was formed of cribs. The dimensions and general form of cribs were governed by the profile of the bank immediately at the point where the dike is located. The dike under consideration contained but two cribs, the lower one 212 ft. long by 32 ft. wide and 8 ft. deep, of uniform section; the upper one, 170 ft. long, 16 ft. wide and 8 ft. deep. The longitudinal axis of the cribs was 90 ft. below the up-stream edge of the mattress. The dike was finished with as nearly a uniform surface as possible, having a slope of 4 to 1 from the river end to the top of

the graded bank. The cribs were each right rectangular prisms in general form. Where four or five were required, the top one was 16 x 8 ft., the next 32 x 8 ft., the next 48 x 8 ft., and so on, the last one sometimes being constructed to conform to the irregularities of the bottom, but having a uniform top surface.

The details of crib construction were as follows:

A framework of poles and willows was laid on the floating ways on which the mattress was constructed, in 8-ft. squares, to the required length and breadth. On this frame layers of brush, both longitudinal and transverse, were placed, and on these another frame similar to the first, then other layers of willows followed by a frame, and so on until the crib had attained a thickness of 3 ft. Then the frames and intermediate layers of brush were thoroughly sewed together with No. 12 wire and 9-in. spikes, wire being left projecting far above the last frame to fasten the lower to the upper cribwork. When the construction had reached this stage the partly finished crib was launched from the mat barge and placed over the mattress on which it was to rest. It was then fastened by lines to the mooring barges, which held it in place, and also connected with the shore by shackle lines leading well up the river.

At the intersections of the frame poles, 8 ft. apart, upright posts 9 ft. high were then securely fastened, and to these, as well as to the lower frame, the crib was built, pockets or compartments being formed in each 8-ft. square, the shape of an inverted frustum of a pyramid, by dropping back one pole as each layer of brush was laid above the bottom 3-ft. foundation. When finished the crib was 8 ft. deep, with pockets 8 ft. center to center, about 5 ft. deep, and each capable of holding about 3 cu. yds. of loose rock. The whole crib was securely held together throughout with spikes, wire and $\frac{3}{8}$-in. cables. The process of sinking was practically the same as in the case of the mattress. After the crib was securely fastened to the bank by cables placed carefully over the mat in the desired position, and after being checked up well by slip lines to the mooring barges, rock was thrown into the pockets until they were filled. When the weight had considerably more than overcome the buoyancy of the structure, the strain on the lines being very great and the crib in a sinking condition, then the slip lines were rendered slowly, care being taken to keep the strain uniform until the crib settled in its place on the mattress. In the case

of the second crib, it was built 16 ft. narrower, as before stated, and sunk in the same manner on top of the first one, thus forming a sill 16 ft. high.

After the mattress was constructed and sunk, before the cribs were started, the portion of the bank above the mat and water line was graded to a slope of 4 to 1, preparatory to covering it with a revetment of willow and stone, which was connected with the submerged mattress.

The grading was done with a sluice as before described.

After completing the grading, the revetment was laid; first a framework of poles in 8-ft. squares, then two diagonal layers of brush placed on it, then another framework corresponding to that below, and the entire mass tied together securely with wire and spikes. On this, stone was laid close enough to hide the brush.

When the revetment was finished and the mattress and submerged cribs sunk, the upper end of the latter was joined to an upper-bank crib which carried the regular slope of the top surface of the dike to its intersection with the upper bank.

This upper-bank crib, though constructed on the ground, was in form similar to those made on the ways, and formed but the continuation of the subaqueous work.

The brush for the work cost per cord....	$1 27½	and	$1 09
The poles, per cord	1 47½	"	1 48
The rock, per cubic yard.....	1 50	"	1 95

All of the material was delivered, the brush and poles at the contractor's camp, and the stone at the work on board of government barges, for that price. The total money expended in building two dikes was $24 037 76. More cables and other materials were used in these dikes than had formerly been the custom. The field cost and material used in their construction are shown in Appendix A.

The total cost of the dike was: Foot mat, $3 830 49; crib No. 1, $1 698 73; crib No. 2, $871 55; revetment, $3 167 82; shore crib, $92 69; total field cost, $9 661 28. The gross cost of the entire work was considerably greater than the net cost as shown in this dike would indicate, because of the difficulty of obtaining good labor during that season, the run of drift, the remoteness from the base of supplies, and the excessive amount of material used in the first dike constructed, the cost per unit being much in excess of that of the second dike.

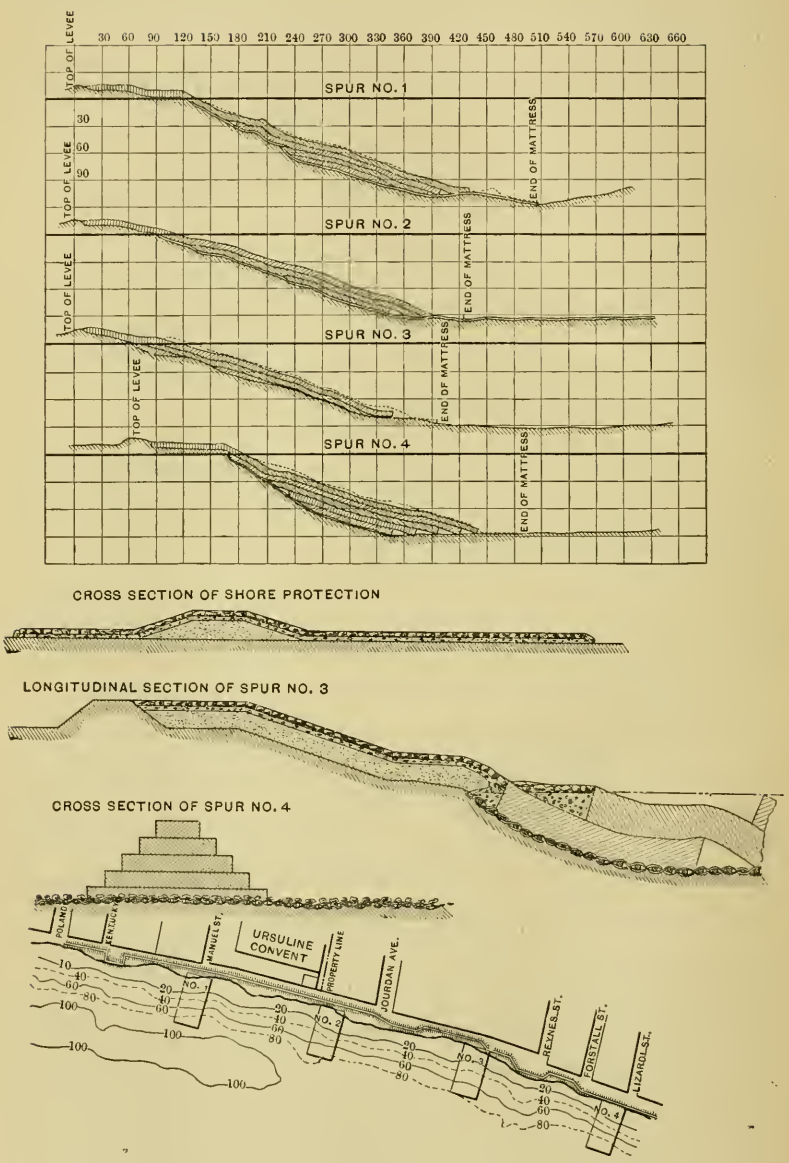

TOP OF LEVEE
30 60 90 120 150 180 210 240 270 300 330 360 390 420 450 480 510 540 570 600 630 660
SPUR NO. 1
30
60
90
TOP OF LEVEE
END OF MATTRESS
SPUR NO. 2
TOP OF LEVEE
END OF MATTRESS
SPUR NO. 3
TOP OF LEVEE
END OF MATTRESS
SPUR NO. 4
END OF MATTRESS
CROSS SECTION OF SHORE PROTECTION
LONGITUDINAL SECTION OF SPUR NO. 3
CROSS SECTION OF SPUR NO. 4
POLAND
KENTUCKY
MANUEL ST.
URSULINE CONVENT
PROPERTY LINE
JOURDAN AVE.
REYNES ST.
FORSTALL ST.
LIZARDI ST.
NO. 1
NO. 2
NO. 3
NO. 4
10
20
40
60
80
100

FIG. 18.

The cost of continuous repair revetment work at Bolivar Landing during the season of 1889 was $10 68 per square.

New Orleans Spur Dikes.—In 1890 the construction of spur dikes at New Orleans was continued. There the dikes were much larger than at other points. In that portion of the river, this particular kind of bank protection is especially well adapted to the existing conditions. As before stated, the small variation in elevation of water surface, as compared with points above Red River, the less rapid caving and the more uniform flow, all produce a condition favorable for the adoption of that method.

One of the dikes built this season was the largest ever placed in the river. It contained about 3 400 cords of willows, 80 000 ft. of lumber, 2 000 tons of rock, 5 500 lbs. of wire, 60 kegs of nails and spikes, and 8 000 lbs. of iron rods and chain. Its length was 430 ft., and height 60 ft. The depth of the water at its outer edge was 152 ft.

The dikes were constructed of sawed lumber frames in place of rough willow pole frames, as in the upper-river work, and they were formed of cribs in some cases five and six tiers high. Fig. 18 shows cross-section of some of these spurs of 1890.

This season a departure was made from the plan before adopted, which changed the dikes from purely submerged structures to a form somewhat similar to those used at Memphis and Greenville. This consisted in carrying the dike above low water by connecting it with an earthen levee with a crown of 10 ft. and slopes of 3 to 1, the bank being first graded to a gentle slope. On this levee rock was placed to a thickness of 9 ins., forming a pavement covering the levee and extending 20 ft. above and 50 ft. below it (see Fig. 18).

The average field cost of these dikes in 1890, not including administration, repairs, etc., was $7 15 per square for mattress work and 3.6 cents per cubic foot for cribwork. These spurs average 1 000 ft. apart.

The standard revetment by this time had become generally adopted, and consisted of continuous revetment and dike construction. The standard revetment as used in 1890 in the upper districts is described in detail in the report for 1891 of the Chief of Engineers of the United States Army. As the woven mattress and willow upper-bank revetment had reached its most perfect form, and was shortly to be replaced by an entirely different kind of willow work for the subaqueous portion and paving of the upper bank, a description of it will

be given as illustrating the highest development from the crude methods of 1882 of this type of protection work.*

Construction of Standard Bank Protection, 1890.—After staking out the top of the slope to a grade of 4 to 1, measuring from the low-water line, the bank is cleared of all trees and trash 50 ft. back of the top.

When the clearing has been carried ahead a sufficient distance, one or two hydraulic graders are started and the bank sloped in the manner heretofore described. "After grading, the bank is dressed by hand, all holes being filled up with brush and dirt."

Either after or before grading, the latter being preferable, all snags are removed from the bank in the reach in which the mattresses are to be placed. Two methods are resorted to, blasting and pulling with steamboats. The snag boat method is less expensive, but more satisfactory if followed by the submarine diver.

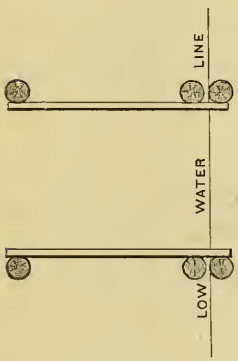


FIG. 19.

When a considerable reach of bank has been cleared, the snags removed, and the grading completed, an abutment is constructed consisting of two piles driven 15 ft. apart and parallel with the bank, and 10 to 15 ft. inside the low-water line. Out from and directly opposite these, at the low-water line are driven two clusters, with two piles in each cluster (see Fig. 19). An inclined brace connects each cluster to the pile behind it. These piles are left about 10 ft. above medium low water. The braces are sawed off flush with the outer face of the twin piling, to allow for mooring barges.

On this abutment, when the mattress barges are swung into place, the inside edge of the mooring barges rest, and are held out to the required position. Below this abutment, on the zero line, single piles are driven 100 ft. apart for the full length of the required mattress, the top of these piles being left, as in the case of the abutment clusters, about 10 ft. above the medium low water. By means of this piling the mattress is kept over the zero line during a limited fluctuation of the water surface, while being constructed and sunk. After the mattress is on the bottom, all these piles are cut off or pulled out, as they would otherwise form obstructions to navigation.

* See the paper by Mr. A. G. Nolty, assistant engineer, on details of construction of bank protection works, as practiced at Plum Point Reach in 1890. "Report of the Mississippi River Commission," 1891, p. 3606.

"While the abutment is being constructed two mooring barges lashed end to end are placed alongside of the bank above the abutment. Outside of these barges, parallel with them and with the ways touching the gunwale, is placed the mattress or weaving barge." The head lines connecting the mooring barges with deadmen or trees on the bank consist of six wire rope cables, the up-stream one 1¼ ins. in diameter, the next two 1⅛ ins. in diameter, and the last three 1 in. in diameter, as shown in Fig. 20. These cables have an eye spliced in one end, which is used for fastening them to the cavel of the mooring barge. On the up-stream edge of the latter the timber heads have heavy iron bands forming eyes through which the eye of the cable is brought and fastened by a 1¼-in. iron bolt.

While the mooring lines are being run, the head of the mattress is under construction.

"Hardwood poles, as large as can be conveniently handled by a gang of men and reasonably straight, are laid in two lines on the ways over and parallel to the inner gunwale. These poles lap each other 10 to 15 ft., the two lines breaking joints. Where they lap they are spiked together with 8¼-in. spikes every 2 ft., and also tied together with No. 12 galvanized wire at intervals of 10 ft., except at the laps, where two ties are made. This line of poles is equal in length to the width of the mattress."

Upon these poles, at intervals of about 7½ ft. and at right angles to them, the butt ends of the weaving poles are securely fastened with two 8 x ¼-in. spikes and a lashing of No. 12 wire. Upon these latter another set of poles similar and parallel to the first are placed, spiked and wired, and the whole head thus formed securely fastened with wire.

"The weaving poles are live willow or cottonwood brush reasonably straight, and 4 to 6 ins. in diameter at the butt, and from 25 to 30 ft. long.

"To facilitate weaving all knots are trimmed off and the top and bottom quickly smoothed with a draw knife. A cable made of eight strands of No. 12 wire is fastened around the head of the mat at every third weaving pole and run up alongside of it, being fastened thereto by two staples. These cables are 24 ft. long, with an eye in one end, to which, after each shift of the mat, a new length is looped in weaving. Ten continuous cables are thus formed in the mat, greatly strengthening it longitudinally."

While the head is being formed the mattress head lines are being run out, and when it is finished they are fastened to it. These lines are five in number; one near the outer edge of the mattress, 1⅛ ins. in

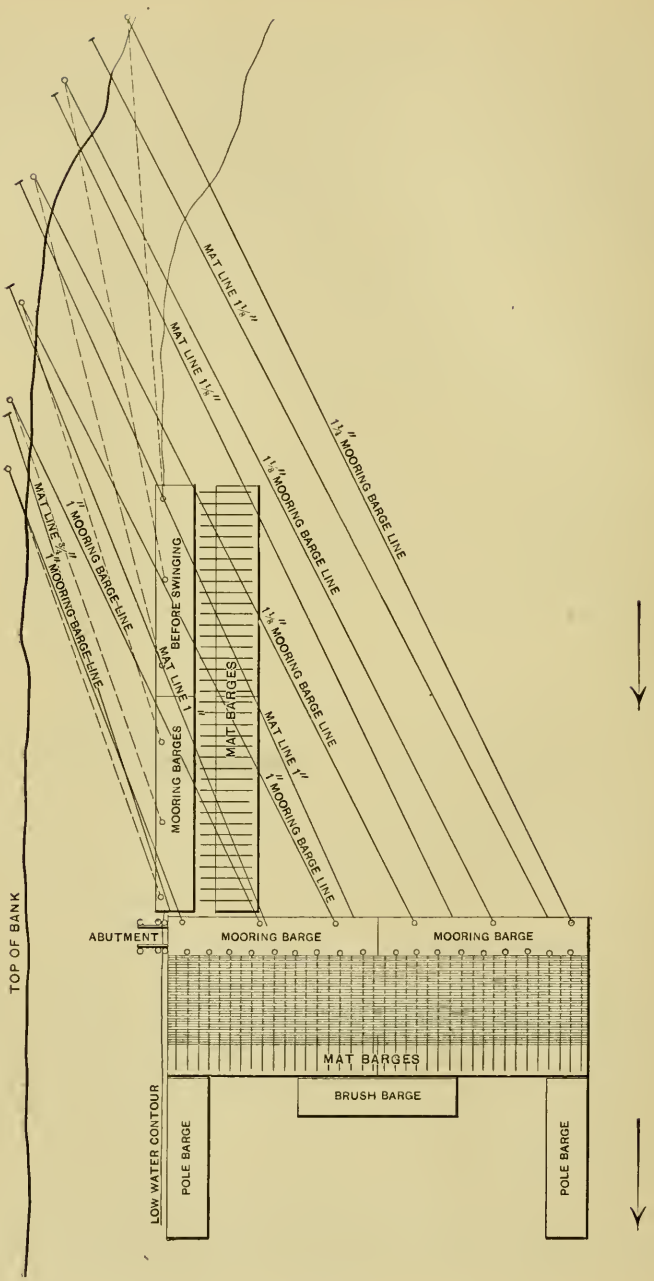

MAT LINE 1¼"
1¼" MOORING BARGE LINE
MAT LINE 1⅛"
1⅛" MOORING BARGE LINE
MAT LINE 1"
1" MOORING BARGE LINE
MAT LINE ⅞"
⅞" MOORING BARGE LINE
BEFORE SWINGING
MOORING BARGES
MAT BARGES
TOP OF BANK
ABUTMENT
MOORING BARGE
MOORING BARGE
MAT BARGES
BRUSH BARGE
LOW WATER CONTOUR
POLE BARGE
POLE BARGE

FIG. 20.

diameter; then, at equal intervals to the inner edge, one, $1\frac{1}{8}$ ins.; two, 1 in.; and one $\frac{3}{4}$ in. They lead from shore fastenings, as in the case of the mooring lines, and pass under the mooring barges, being fastened to the mat-head loop by a pin shackle. The mat-head loop is made of 2-in. manilla rope fastened to the head and running back into the mat at least 25 ft.*

"The brush used for weaving is live straight willow of any length above 25 ft., and from 2 to 4 ins. at butt.

"A full complement of men for a 200-ft. wide mat is 54, under a foreman, divided into three equal gangs, each under a master laborer. Each gang consists of five men on brush barge passing brush to weaving party, twelve men in weaving party, and one man on mat mauling brush tightly into place as the weavers push it down. Each gang builds one-third the width of the mat.

"The brush barge is placed outside (below) the mattress barge about midway between the two ends, and a barge loaded with poles is hung to each of said ends.

"A coil of 2-in. rope is placed on each end of the mat barge, and the free end fastened to a timber head on the mooring barge. The skids on the ways are well slushed with axle grease or tallow, to reduce the friction in weaving and launching. In a 200-ft. wide mat, 27 weaving poles are used. In weaving they are close to the skids or ways and at the upper end rest in brackets, bolted to the upright posts on the mattress barge.

"The brush is passed from the brush barges to the weavers who work in pairs, and is woven by them into the mattress. The butts are placed over one weaving pole and 2 ft. beyond, being woven at the other end over the next pole, under the third, over the fourth, and so on, the light ends being always left on top."

A strip 5 ft. in width is woven thus, in the next 5 ft. the butt is reversed, and so on, the butts changing directions every 5 ft. When the mattress is woven to within 2 ft. of the end of the poles, giving about 22 ft. length of mattress, it is swung in position with the accompanying barges.

The entire floating plant is swung on two lines, a 2-in. manilla rope on the outside and a $1\frac{1}{2}$-in. manilla rope on the inside of the mooring barges, having one end fastened to the shore (see Fig. 20). The head lines on the barges and mattress are slackened until the barges take a position nearly normal to the shore, with their inside edge resting on the pile abutment.

* The number and strength of lines depend on the velocity of current, the stage of water and amount of drift running. In the lower districts more fastenings, as a rule, are used than as shown above.

"The slack in the mooring barge cables is now taken in from the bank and the strain equalized. They are then fastened permanently with clamps furnished for that purpose, the mattress head lines being treated in the same manner."

To prevent the mattress sliding too far when launching a finished shift, five 1¼-in. manilla lines 100 ft. long are fastened equidistant on the mat, and pass underneath it to cavels on the ways. They are slackened off in launching. Their fastenings on the mattress can be shifted to economize rope.

"The slip lines are 1¼-in. manilla rope about 125 ft. long, there being 18 to a 200-ft. mat. Each has an eye large enough to pass over a timber head. This eye is placed over one of a line of timber heads on down-stream edge of mooring barge, the free end being passed under the head of the mat and up again on its down-stream side, and then hauled taut and fastened on its timber head. The mattress head is thus hung in 18 slings. When these slip lines are all adjusted the mat is further launched until the down-stream edge is over the up-stream gunwale of the weaving barge. The office of these lines is two-fold; they hold the head of the mat up, preventing the current and drift from forcing it under, and they are used to lower it to the required depth in sinking.

"When one entire shift (about 22 ft.) is launched, a new set of weaving poles is spliced to the projecting ends of the first set, the butts of this set being spliced to the tops of the preceding one, after having been pushed into the mat about 3 ft., thus making a lap of 5 ft. Two 8 x ¾-in. spikes and two wire lashings are used to fasten each splice. This is continued as described to within 2 ft. of top of second set of poles, when another launch is made, and so on, until the full length of the mattress is obtained."

The mooring barges were swung to a position not quite normal to the shore, but as the mattress is constructed, the greater pull on the outside lines and careful adjustment of all the strains brings them into the perpendicular.

"As soon as three shifts of mat are launched, the construction of a top grillage or framework is begun, consisting of a line of poles laid over the weaving poles and parallel thereto, lapping each other, butts to tops, from 6 to 8 ft. and wired to the weaving poles every 4 ft. by lashings 2 ft. long, made of two strands of No. 10 wire; transverse poles 8 ft. apart for the first 100 ft., and thereafter 16 ft. apart are placed in similar manner and fastened to the longitudinal ones at the intersections by 2-ft. lashings, made of four strands of No. 12 wire. This grillage is for the purpose of forming cribs, to retain the stone on steep slopes, and it also strengthens the mattress. The first set of

transverse poles along the inner edge are hardwood, are only 8 ft. apart throughout the length of the mat, and are used to connect the shore mat to the river mat.

"The construction of the shore mat, which is expected to progress with the river mat, is carried on by a gang of from 30 to 40 men under a foreman and one master laborer. The space to be spanned in building the shore mat varies with the stage of water from 0 to 60 ft. In its construction a small flat boat is generally used from which to work. Hardwood poles of the size of the weaving poles are lashed to the river mattress with three No. 12 wire lashings 2 ft. long and spiked with two 8 x ¼-in. spikes. Willow or cottonwood poles are spliced to these until they reach up the slope about 40 ft. Alongside and fastened to each of the hardwood poles is a cable made of eight strands of No. 10 wire, one end of which is fastened to two of the adjacent weaving poles, and the other to the willow poles extended on the slope. Upon the transverse poles are laid longitudinally willow or cottonwood poles 8 ft. apart, beginning with the first set about 4 ft. from the edge of the mat. The latter poles are wired to the former at their intersections with lashings of No. 12 wire 2 ft. long. The longitudinal poles are carried on lines 8 ft. apart up to the top of the slope, and on their lower side, 8 ft. apart, are driven stakes 2½ ft. above the ground, to the top of which is fastened loosely a lashing of No. 12 wire 2 ft. long, whose bight has been first passed under the pole. These stakes are used down to the pole nearest the water edge. Upon this framework is laid willow brush diagonally with the butts toward the top of the slope and breaking joints throughout, except at the top of the slope. A second layer of brush is laid upon the first in the opposite direction, butts pointing to top of slope and breaking joints, the direction of each layer making a right angle with the other. On top of these layers of brush a second pole framework, fastened in the same manner as the first, is placed and fastened down firmly by the lashings tied to the stakes. As fast as the river and shore work is finished transverse cables made of eight strands of No. 10 wire and 60 ft. long, with an eye in each end, are run across the entire width of the mat every 16 ft., carried to the top of the bank, hauled taut and fastened to trees, stumps or deadmen. They are also fastened to the mat every 16 ft. with wire lashings."

When 400 or 500 ft. of river mattress is made, longitudinal cables* are run out from the mooring barge, one ⅝-in. cable close to the outer edge of the mat, another about 30 ft. inside of the first, the third ½ in. in diameter and 37 ft. from the second, a fourth of the same

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* "These cables, which are a continuation of the mattress head cables, are furnished ready made, of the following diameters, ⅝, ½ and ⅜ in., each 1 200 ft. long. They come in coils, and before using are mounted on reels."

dimensions 38 ft. inside of the third, and the fifth and last, $\frac{3}{8}$ in. in diameter and 42 ft. from the fourth, or 45 ft. from the inner edge of the mattress. All are fastened to the mattress every 16 ft., perfectly taut throughout the mat and secured to its head and foot when finished.

The cables are mounted on reels, the outside or $\frac{5}{8}$-in. one being placed in position first.

"The whole cable is reeled off, but the inner end kept on the mooring barge where the reel is set up. After being run down towards and close to the weaving barge, the spare cable is coiled down and tied for further extension. Just above the coil the cable is fastened to the mat with a wire lashing and a specially designed clamp. A tackle is then fastened to the free end and the cable hauled taut by 10 men on the mooring barge. It is then fastened to the mat every 16 ft. with a clamp and lashing. The second $\frac{5}{8}$-in. cable is next run out and fastened, where the second head cable takes hold, then follow the two $\frac{1}{2}$-in. and finally the $\frac{3}{8}$-in. cable. As weaving progresses, the cables are extended until the mat is finished, when the ends are securely fastened around the foot of the mat. After the cables are run, hauled taut, and fastened to head of the mat, all additional hauling is done from lower end of the mat."

If the mattress is to be continuous and very long, when 600 ft. have been woven ballasting can begin; but when made in lengths of from 800 to 1 000 ft., the mat is generally completed before being loaded. The ballasting is accomplished by placing a loaded stone barge outside of the mat, hanging it to the mooring barge by 2-in. manilla rope and holding it close to the mattress by breast lines.

Cottonwood planks 16 ins. wide, 3 ins. thick and 24 ft. long are laid from the barge to the mat, and are continued across the mat in 18-ft. lengths 10 x 2 ins. in section. From two to four of these runs are used and from 10 to 15 men employed on each run. These men wheel out the stone in barrows and dump it along the transverse poles, loading the entire floating mat until only the poles are above water, being careful to load the 50 ft. next the bank heavier than the rest. There is one foreman and one or two master laborers to this gang. The men on each run go out and return together. When the length of the barge is ballasted across the mat, the barge is dropped one length and the planks changed to the new position. The shore work is not ballasted until the river mat is sunk, except where stone is piled on the bank, in which case the shore work, as well as part of the river mat, can be ballasted from the bank.

"As soon as the mattress is completed and ballasted, a loaded stone barge is brought up to the mooring barge; a line is run from its head to the shore capstan of the mooring barge, and another from its lower end to the outer capstan. A man is stationed at each slip line who obeys only the word of the general foreman. Stone is thrown on the head of the mattress, and as soon as the strain on the slip lines is considerable, they are carefully slacked for a short distance. The men at the in-shore capstan begin to haul the head of the stone barge gradually over the submerged mat. The lower end is also gradually hauled up, the object being to bring the stone barge squarely across the mat, stone being continually thrown over on the mat and the barge hauled over until this is accomplished. The line from the down-stream end of the barge is then quickly shifted to another capstan as the barge proceeds in shore."

In the mean time another stone barge is brought up and placed end on the one nearest the shore and securely lashed to it, thus forming a line of barges just below the mooring barges, parallel to them and floating over the head of the mat which is about 10 ft. below the surface, but kept from sinking further by the slip lines.

"In the mean time one coil of 2-in. rope has been placed on the inner end of the first and one on the outer end of the second barge, and the end of each fastened to a timber head. A long 1-in. line also runs from the inner end of the first barge ashore, where a gang of men will be ready to haul the in-shore end of the barge down stream, should the water near the shore be without current. A steam towboat now makes fast with one line to the outer end of the outside barge. A man is at each slip line. A line of men is distributed along each edge of the stone barges. The men on shore have hold of the 1-in. hauling line, the steamboat hanging from and close to the outer edge of the stone barges, and lying with the current. The general foreman is on the mooring barges, watching the slip-line men, and everything is in readiness for the final operation.

"The general foreman now gives the word to throw the stone, and as soon as the slip lines show the required strain, he orders the linesmen on the stone barges to slacken away. He then gives the word, and the slip lines are let go simultaneously. The mat settles quickly at the head, and as the stone barges are dropped down squarely over the body of the mat, the stone being rapidly thrown on to it, it also gradually settles to the bottom. When the required amount of stone is unloaded on the mattress and it is securely on the bottom, the mattress head lines are taken up by a sailor gang, by hauling on the pin lines that are loosely fastened on the mooring barges. These lines pull the pins out of the shackles and set the cables free from the mat. The mooring barges are then allowed to swing to the bank, and all cables are reeled up on their drums, being first washed and oiled."

In the case of a bank which is caving so rapidly that the grading cannot be accomplished before the mattress is built, a foot mattress is constructed joining the main mattress with the bank, the upper bank work being deferred until the main mattress is sunk.

"The abutment and main mattress are constructed as described, except that the hardwood poles along the inner edge are omitted, and only the regular top grillage is laid. After the mattress is sunk, the bank is graded and the shore work laid down to the water edge. A connection or foot mat is then made, to connect the shore work with the submerged river mattress, as follows :

"The weaving barge is brought up alongside and parallel to the bank, with its up-stream end over the head of the mat, and a mattress built like the river mattress, except that it has a lighter head and no mooring barges are used. Instead of the heavy wire cable, wire strand is run from the head up to the top of the bank and fastened there. Five $\frac{5}{8}$-inch cables are used for every 200 ft. of mattress. Two or three cables are also run from the up-stream edge of mat to shore, to prevent current from taking it down stream. The head of the mat when launched off will lap 4 or 5 ft. over the shore work. To launch a shift, the barge is sparred away from the bank. When completed and sunk, the foot mat laps over the river mat, also, 5 ft."

Plate III, Fig. 2, shows the woven mattress under construction on the ways before the top grillage or frame of poles is added; Plate IV, Fig. 1, shows the woven mattresses under construction, partly poled on top, with the bottom frame of the upper-bank revetment in the foreground, and Plate IV, Fig. 2, shows the commencement of the operation of mattress sinking.

Work During 1890–92.—At Ashbrook Neck, during the season of 1890, mattresses were made 300 ft. wide. The bank was cleared 230 x 6 000 ft., or 31.6 acres. The bank was sandy, and was graded by a hydraulic grader, and dressed with shovels to a slope varying from 3 to 1 to 5 to 1, and a height of about the two-third stage. From the shore mat to the top of the grade was paved with rock to a depth of 10 ins.

The distribution of material was as follows : Brush, 0.708 cord per square ; poles, 0.136 cord per square ; stone, 1.08 cu. yds. per square.

The work consisted of 3 036 lin. ft. of main mat, equal to 8 688 squares ; 980 lin. ft. of shore mat, equal to 619 squares ; and 2 820 x 65.3 ft. of upper-bank revetment, equal to 1 843 squares.

The cost per square was $7 69, and per lineal ft., $30 42.*

* This may include the expense of a small amount of abortive dike work, undertaken at the beginning of the season.

FIG. 1.

FIG. 2.

A survey in this year of New Orleans Harbor proper showed but a small change in the bank, except a slight moving out of the 100-ft. contour. There had been no fill between the dikes, but, on the contrary, a noticeable scour. In Carrollton Bend the work had not checked the caving.

The field cost and dimensions of the dikes this year were as follows :

Dimension of section.	Greatest height.	Cost.
415 x 140 ft.	45 ft.	$17 632 05
355 x 130 ft.	50 ft.	13 008 69
400 x 120 ft.	40 ft.	10 901 79

The foundation mattress for these dikes was dissimilar to those previously described. They were made in sections, afterwards joined by wires, and not of the woven type. They were framed mats, the bottom layer of willows being securely nailed to the bottom frame. Three layers of willows were laid, the bottom and top parallel to each other. The mattress was built in four sections—three, 100 x 130 ft., and one, 55 x 130 ft. The different sections were placed in position alongside of a lowering barge and fastened together with No. 10 galvanized wire, making a mattress 355 x 130 ft.

The up-stream frames were built of 3 x 6-in. lumber and strengthened at each toggle pin by three lines of iron rods, one running straight, and one on each side of the pin diagonally, across the mattress, crossing the frames, and fastened with wire to each frame. The frames were placed at right angles to the bank, and top poles were placed longitudinally about 16 ft. apart, to keep ballast from sliding off on steep slopes, and also to give greater strength.

The dimensions of one of the mats were 355 x 130 x 1.75 ft., or 461 squares. The distribution of material and cost was as shown in Appendix A.

There was but little change in crib construction. The cribs placed on the previously mentioned mat were in number and in dimensions as follows: Crib No. 1, 120 x 56 x 6 ft.; No. 2, 152 x 48 x 6 ft.; No. 3, 170 x 40 x 6 ft.; No. 4, 200 x 32 x 6 ft.; No. 5, 274 x 24 x 6 ft.; No. 6, 310 x 16 x 6 ft.

There was 232 512 cu. ft. of material in the cribs, the cost per cubic foot being 3.57 cents. The total cost of the dike was $13 008 69.

Other spurs built at the same time having approximately the same distribution of material cost as follows: Mattress, $9 95 and $9 48 per square; cribwork, 3.58 and 4.31 cents per cubic foot. The average cost of cribwork was 3.59 cents per cubic foot, and of mattress, $9.987 per square. It will be noticed that the cost of mattress per square far exceeds that of the woven mattress in the upper districts, and is much more expensive than the woven mattress made in New Orleans Harbor in 1890.

During the high water of 1891 the Memphis dikes settled 4 ft. at outer ends on an average.

At Fletcher's Bend it was found that the interrupted revetment, heretofore referred to, was not a success. The water attacked the bank in the unprotected intervals, causing it to cave, and it was found necessary to revet them, thus making the work continuous and demonstrating the non-feasibility of interrupted revetment in that portion of the river.

In Ashport Bend in 1891 piles were driven 50 ft. apart along the zero line. Against these the inner edge of the mattress rested, being fastened to them by yokes made of $\frac{5}{8}$-in. wire strand running across the mat and connected with it every 16 ft. After sinking the mattress the piles were cut off as close to the ground as possible.

This season, in Hopefield Bend, the cost of hydraulic grading, including clearing and dressing bank was $6\frac{3}{4}$ cents per cubic yard. The distribution of material in mattress work was: Brush and poles, per square, 0.77 cord; stone, per square, 0.71 ton.

For the connecting mattress: Brush and poles, per square, 1.40 cords; stone, per square, 1.27 tons.

The upper bank was paved at a cost of $6 12 per square. The approximate cost per lineal foot of completed revetment 200 ft. wide below zero line, with bank paved to the two-thirds stage, is given by the engineer in charge of the Hopefield work as follows: Clearing and grading, $1 59; subaqueous mattress, 200 ft. wide, $7 02; connecting mattress, 60 ft. wide, $3 36; paving, $4 59; other expenses, $3 01; total cost, $19 57.

At Ashbrook Neck in 1892 the mattresses were made from 250 to 300 ft. wide, and the bank was graded to a 4 to 1 slope. Brush revetment was carried up 5 ft. above the inside edge of the mattress, and the upper bank was paved with 10 ins. of riprap to the two-thirds stage, or 32.8 ft. on the Arkansas City gauge.

The cost of clearing the bank was $71 50 per acre; grading, $1 21 per lineal foot; dressing grade, 47 cents per lineal foot. The total cost of grading was $1 68 per lineal foot. The cost of mattress per square was $4 69, and the cost of the shore mattress was $5 33. The bank revetted was 4 460 lin. ft., and the cost $30 83 per lineal foot. The average width of the work from top of slope paving to outer edge of mattress was 419 ft.

The upper-bank work took 2.89 cu. yds. of stone per square.

The cost of this work and distribution of materials were as shown in Appendix A.

At Greenville continuous mattresses were sunk 300 ft. wide, and the upper bank graded and paved to the 30-ft. stage.

Cross cables were used extending from the outer edge of the mat some distance up the graded bank, and woven into the mattress; also from eleven to fourteen longitudinal cables, each formed of 19 strands of No. 12 wire, in a mat 300 ft. wide. To hold the mat in place during construction and sinking, ten steel 1-in. wire cables with suitable shackles and 1½-in. pins were used. These cables were in 400-ft. and 800-ft. lengths, with shore ends securely fastened. The mooring barges were held in place by four or six 2-in. manilla lines. The usual slip-lines were used, and at one or two localities Chinese anchors were resorted to.

The mattress cost $4 50 per square, and the material used was: Brush, 0.70 cord per square; poles, 0.13 cord; stone, 0.64 cu. yd.

On the upper bank 3.54 cu. yds. were used per square. The cost of this work and distribution of materials were as shown in Appendix A.

The cost of this entire work during 1892, including administration, plant repairs, etc., was $29.517 per lineal foot.

At Louisiana Bend during the same season the mattresses were made 270 ft. wide. The upper-bank work was very light in places. The material used per square was as shown in Appendix A.

The total extent and cost of the work were:

	Squares.	Per square.
Channel mattress	14 294	$5.045
Pocket mattress	3 722	6.239
Upper revetment brush	2 547	7.059
Paving	2 547	8.329

The total cost of work here during the season was $140 033 78; the amount accomplished, 23 110.5 squares, at $6 06 per square. The cost per lineal foot was $26 49.

The cost of mattress and crib work in New Orleans Harbor in the season of 1891–1892, the work being similar to that of the previous season (frame mattresses in 100 x 120-ft. sections joined together) was: Mattress, $9 28 per square; crib work, 3.7 cents per cubic foot.

Later Work of Commission.—Notwithstanding the fact that the protection work had been increased in strength from the very beginning, the whole history of the work being one of continued increase in dimensions and strength, it was still found unequal to the strain put upon it at certain localities. Surveys and subaqueous measurements and observations made in 1892 and 1893 showed a deepening and deterioration at the outer edge of the river mats at nearly all points. In the report of the Commission for 1893 there is the following statement:

"There has been a deepening from scour along the outer edge of the mats. In some cases the mat has adjusted itself to the new condition, as was intended, while in others the test of its flexibility has been too great and faults have occurred. In some places also there has been settlement in the middle of the mats rather than along their edges, indicating that greater thickness or density is required in very exposed situations. Defects have also been found between the low-water mats and those built on the graded bank.

"The mattresses used in the lower Mississippi for five years past have been the heaviest and widest ever made for like purpose in the history of engineering. To build and sink them in the deepest and swiftest stream upon which such improvement has been attempted is an undertaking of extreme difficulty. It could not have been done successfully in the earlier stages of the improvement."

In view of these facts it was determined to further strengthen the mattress and to make 50 ft. of the outer edge more flexible, so that it would conform to the new scoured slope without being broken; also to give greater stability to the junction of the subaqueous work and the upper-bank revetment. It was claimed that the protection work up to that time had accomplished the purpose for which it was designed, and as it had proved to be the correct method of improvement the necessity for making it permanent arose.

When a reach or bend is not protected, the water at medium high stages and on a rising river exhausts much of its force in cutting down

the bank and deepening the bed. When the bank is protected, the currents pass over the protected portion, unable to expend their energy in work other than the infinitesimally small abrasion of the willows and rock, the water thus gathering increased velocity and greater scouring force. At the outer end of the revetment, where the current generally attains the greatest velocity, this force finds material on which it can act with greater facility, thus deepening the bed immediately at the outer edge of the mattress, undermining it and causing it either to sink and adjust itself to the new condition or break.

It is probable that this will always be the case up to certain limits of depth and rate of flow when, if the mattress work is sufficiently strong, equilibrium will obtain and the work be permanent unless attacked by forces within the bank itself and not produced by the action of the river water. Increased depth and settling at the end of the subaqueous work steepens the slope and increases the tendency of the bank to slough in badly drained localities.

The foot of the mattress should always reach beyond the greatest depth of the inshore thalweg, if possible within specified limits of economical practice.

The revetment built in accordance with the new requirements of flexibility and strength cost as follows, including all expenses except office: *Ashport Bend.*—River mats, per square, $4 27; connecting mats, $8 17; pocket mats, per square, $5 90; paving, per square, $10 11; grading, per square, 3¾ cents. *Daniel's Point.*—River mattresses, per square, $5 09; connecting mattresses, per square, $7 81; paving, per cubic yard, $1 99; grading, per cubic yard, 6 cents.

The work in 1892 in New Orleans harbor consisted of placing mattresses in the spaces between the dikes where caving had occurred or was threatened. These mattresses were practically the same as those constructed and placed under cribs the year previous, and cost $8 92 per square.

First Fascine Mattress.—At Daniel's Point in this year a new type of mattress construction was attempted, and though the conditions were unfavorable, the plant inadequate, and the labor unaccustomed to the methods, the results demonstrated the great advantage to be attained in substituting the fascine for the woven mattress.

"This new form of mat was constructed of fascines or bundles of brush 12 ins. in diameter and in lengths of 50 and 100 ft., tightly com-

pressed and bound every 3 ft. These fascines were placed at right angles to the bank and formed the woof of the mattress, longitudinal wire strands forming the warp. The top and bottom cables of each pair were clamped together every 3 ft. by long cable clamps. On top of the mattress a grillage of poles was placed and tied down to the frames by galvanized wire."

No further attempt was made in the way of fascine mat construction until 1893, and no estimate is given of the cost of that form of work until that year.

In 1892 the repairs in Hopefield Bend cost as follows: River mattress, per square, $3 65; connecting mattress, $5 89; paving, per square, $11 70; reballasting, per square, $5 50; grading, per cubic yard, 4½ cents.

At Ashport Bend a mattress was made in the same year in which much more brush was used and the longitudinal cables were doubled, one running above and the other below, reversing every 25 ft., thus forming loops 25 ft. long.

The material used per unit on the Plum Point Reach during the season 1892 is given in Appendix A.

The cost of the Third District work per lineal foot was as follows:

Ashbrook Neck.....	$29 07	woven mattress and paved bank.
Greenville Harbor..	27 08	
Louisiana Bend....	27 86	

This averages about $28 per lineal foot. Adding $6 50 for administration, survey, plant, etc., the total cost is $34 50.

Fascine Mat Construction.—The last form of woven mattress, though possessing adequate strength, was not sufficiently thick and compact to prevent the water cutting the bank through the interstices or openings between the brush. It also proved too stiff or rigid to be adjusted to the irregularities of the bank, breaking when undermined, instead of taking the new slope, no matter how steep.

The fascine mattress it was expected would be devoid of both of these defects, and though it was feared the cost of construction would far exceed that of the woven type, it was found that after the employees became educated to the new method, the additional expense was slight.

Two forms of these mattresses were constructed, one with the fascines placed normal to the bank, and the other with the fascines

FIG. 1.

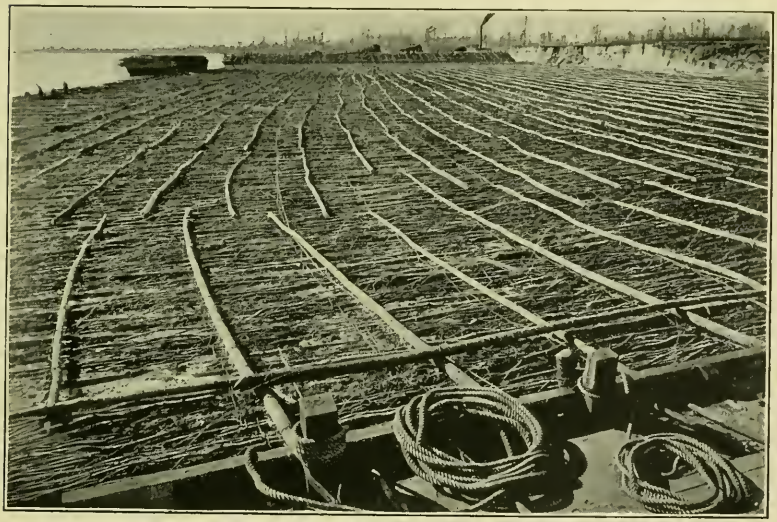

FIG. 2.

parallel to it. The former were first used, but the longitudinal fascine mat was supposed to be more flexible in a transverse direction, taking more readily the irregular shape of the subaqueous slope.

The material and plant used in the construction of these mattresses are similar to those adopted for the woven type, with slight modifications shown in Plate V, Fig 1. The following description and drawings of the details of construction of this new form of mattress, taken from the report of the officer in charge of the work on the upper river in 1894, give very clearly the mode of procedure at that time, which has varied but little at all points up to the present season.*

"The plant required consisted of one set of mooring barges, one set of barges with inclined ways, and one set of fascine barges.

"The barges of each set when placed end to end had a length equal to or greater than the width of the mattress to be constructed. The mooring barges were of the same construction heretofore used with the woven type of mattress, and were moored across the current in the same way. The mattress barges were also the same as heretofore used, with the following additions: Underneath the platform was placed a number of cable drums, one under each inclined way or skid, on which was wound and from which was played out, as the construction of the mattress progressed, the steel wire strand which constituted the longitudinal strength of the mattress, and to which the fascines were attached or woven by the method to be hereafter described. The distance between the drums was 8 or 9 ft. according to the particular barge used. At the highest point of each way an iron sheave was placed, over which was passed the strand from the corresponding drum. The drums were all provided with friction brakes by which a uniform tension was kept on the strands while the mat was in process of construction. The fascine barges were ordinary square-end decked barges on which were erected temporary platforms the same level as the platform of the mattress ways. These barges were put end to end and lashed parallel to and close up to the down-stream side of the mat-ways. On the side of the platform nearest the mat-ways were erected the formers in which the brush was laid, compressed and bound into fascines.

Construction of Mattresses.—"The mooring barges being properly moored in position across the current, the mattress ways, to which had been lashed the fascine barges, were brought in position on the down-stream side and secured to the mooring barges by three lines, one at the center and one at each end. The first step was to construct the mat head. This consists of a bundle of poles, 2½ to

* Report of Captain S.W. Roessler, officer in charge of first and second districts, Mississippi River Improvement, Report of Chief of Engineers, 1894, p. 2859.

3 ft. in diameter, well bound together by wire strand, so as to form a beam of great strength, of some rigidity, but having also considerable flexibility. The poles were of hardwood, 5 to 8 ins. in diameter at the butt, and laid so as to break joints with each other. The beam is as long as the mat is wide, and forms the connecting link between the mooring cables and the mattress. It was moored to the bank by steel wire cables, independent of the moorings of the mooring barges. The number of cables used depended on the width of the mat and strength of the current. Eight cables have been used for a mat 300 ft. wide. They are spaced at equal intervals, being closest together near the out-stream end, where the current is strongest, and farthest apart towards the shore, where the current is

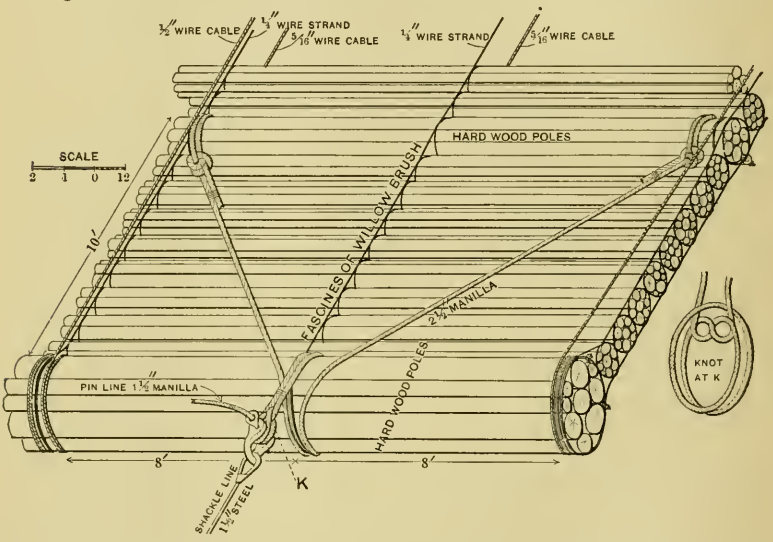

FIG. 21.

more moderate. At each point where a cable is to be attached, a heavy manilla rope, doubled, is wound around the mat head, as indicated in Fig. 21.

"Its free ends are passed over the mat and attached to a second and smaller mat head constructed in the mattress 10 ft. from the head of the mattress. The other end of the rope is connected by an ordinary shackle to one end of its mooring cable, the other end of the cable being then passed underneath the mooring barges to its anchorage on the bank. The wire strands from the drums on the mattress ways are passed over their proper sheaves and wound around and secured to the mat head. The weaving strands, which are ¼-in. steel strand and for convenience cut into lengths of about 50 ft., are also attached to the mat head, one for each of the longitudinal strands upon which the fascines are woven.

"For the construction of the fascines and the mattress the labor is divided into three gangs, one to hand the brush down from the barges, the second to carry the brush to the formers and to construct the fascines, the third to weave the fascines into the mat.

"The second party receiving the brush from the first party carries it to and distributes it in the formers. The brush is put in two layers, one with the tops in one direction and the other with the tops in the opposite direction, care being taken to break joints as far as possible, and especially to prevent two butts from overlapping. Enough brush is put in to make a fascine 10 to 12 ins. in diameter. It is then choked every 8 ft., and bound into a fascine by No. 12 steel wire. The chokers consist of iron chains of suitable length and strength, having one end secured to one side of the former. Each chain is brought over the brush and a lever inserted through a ring in the free end and borne down upon by the weight of one or two men, compressing the fascine while it is being bound with wire. The third party, assisted by the second party, raise the fascine out of the formers, slide it along the skids which connect the former with the top of the mattress ways, and down the latter to its position in the mattress next to the mat head. The weaving strand is then passed over the fascine, down underneath it and up between the fascine and mat head, crossing at the same time the bottom longitudinal strand. It is then put into a 'Haven clamp' and drawn as taut as four to six men can do it with double block and tackle. The strand is temporarily stapled to a pole of the mat head. The second fascine being in place, the weaving strand is passed over the first and second fascine, down underneath the second, and up between the first and second, at the same time crossing the bottom longitudinal strand as before. It is again attached to the clamp and drawn taut as before by block and tackle, and temporarily stapled to a piece of brush in the first fascine. When the strain is first put on the strand, a tap on the pole of the mat head to which it has been temporarily stapled is sufficient to dislodge the staple and allow the strand to completely encircle the first fascine. Details of the method of weaving are shown in Fig. 21. When the ways have been filled the mattress is launched in the usual way and the construction continued. The tenth fascine is made of hardwood brush or poles, and to it are attached the free ends of the large manilla ropes, which are wound around the mat head. The weaving and bottom strands are clamped together by a cable clamp every 10 ft., and at points intermediate between the clamps both strands are stapled to the brush, to prevent the fascines from separating in the process of weaving. On top of the mattress thus constructed are placed rows of poles 16 ft. apart, extending up and down stream. They are lashed to the fascines by No. 7 silicon bronze wire every 5 ft., and at intermediate points by strong steel wire lashings. The object of the rows of poles is two-fold;

first, they prevent the stone from slipping off the mat when it is sunk on a steep slope; and second, by being lashed to the body of the mattress by non-corrosive wire, they prevent the displacement of the brush after the steel wire, weaving strand and other corrosive connections shall have rusted away.

"At the outstream edge of the mat the poles are cribbed up two deep, to better insure against the tipping of the stone when the mat is sunk or when it is subsequently undermined by scour at its edge.

"With a mattress of this type 300 ft. wide, the maximum amount built in a day was 160 ft. with $246\frac{5}{8}$ days' labor. The ballasting and sinking of these mats is performed in a similar manner to that employed in the woven type. In ballasting it is necessary to do the work rapidly and sink the mattress immediately afterward. This precaution is made necessary by the great compactness of the brush fascines and the rapidity with which they accumulate sediment after they have been pressed under water by the ballast. So rapidly do they accumulate sediment that if there is any considerable delay in ballasting, portions of the mattress on which the ballast may be a little excessive may be carried below the surface of the water by the weight of the accumulated deposit before it is ready for sinking.

"Brush not exceeding 3 ins. in diameter at the butt makes a fascine of uniform strength, thickness and compactness.

"One-half and $\frac{5}{16}$-in. strands are used for the bottom longitudinal strands, the $\frac{1}{2}$-in. size being used at the channel edge of the mat, where the current is greatest, and the $\frac{5}{16}$-in. near the shore.

"Smaller strands than these were used in a few of the mats when the supply of the larger strands fell short, but in such cases additional longitudinal strands were stretched over the top of the mat after it was completed and clamped at intervals of 10 ft. to the weaving strand in the same manner as the latter was clamped to the bottom longitudinal cable. The $\frac{1}{4}$-in. strand is used exclusively for weaving. Two forms of clamps have been used, the 'Crosby' and a home-made clamp. Various forms of staples have been used. A staple of the fence-wire type, $1\frac{3}{4} \times \frac{5}{16}$-in., made of No. 9 wire, is a good, cheap and suitable form.

"The oblique position of the mattress head cables to the head of the mattress in the form of construction just described exerts a strong shore thrust on the mat head in the process of sinking. This thrust is taken up by the strong mat head and the mat itself, which together have the requisite rigidity to withstand the thrust without danger of doubling up.

Fascine Mattresses with Fascines Placed Parallel to the Bank.—"Owing to the very much greater lateral flexibility of the mattress in which the fascines are parallel to the bank, it was not deemed safe to have any shore thrusts at all, and the mooring arrangements were accord-

ingly modified. A set of mooring barges was anchored at the head of the mat in the usual way by mooring cables passing obliquely up-stream to deadmen on the shore. A second set of mooring barges was moored in the same way 800 ft. above the lower set. On the deck of the upper barges was placed a continuous line of log deadmen chained to the timber heads, held by the shore cables. These deadmen were to hold the cables mooring the mattress and mattress plant. As the mattress was constructed its mooring cables were attached to it in the manner to be hereafter described, passed underneath the lower mooring barges, carried up stream parallel with the bank and secured to the line of deadmen on the upper barges, thus avoiding any shore thrust upon the mat head. The mattress plant consisted of four mattress ways and their corresponding fascine barges lashed end to end, giving a combined length of 588 ft. The barges were moored parallel to the bank with the ways inclined towards the shore. The fascines were made 588 ft. long, the full length of the mattress. The mattress was constructed by beginning at the water line and extending out stream until the full width of 250 ft. was made. The details of construction were the same as those heretofore described for the mat in which the fascines are normal to the current. To give the mattress more longitudinal strength than the fascines themselves possessed, twelve ½-in. and two ⅝-in. strands, extending the whole length of the mat were built into it in the following manner: When the construction reached the desired point the longitudinal strand was stretched across the bottom strands of the mattress and clamped to them at the points of intersection by common clamps. The up-stream end of the strand was given two turns around the mattress head and carried back and clamped to itself at 130 ft. For additional security the longitudinal strands were connected together by a diagonal system of backing-up straps 100 ft. below the head of the mattress. The mattress head of this mat differed from previous construction, in that it was a kind of jointed spar, designed to adjust to the bed of the river when sunk. The spar was made of 14 cypress logs about 12 ins. in diameter and varying in length from 28 ft. next to the shore to 14 ft. on the out-stream end, all put in place before launching from mattress boats. The logs were placed on top of the mattress and about 3 ft. from the head of the mattress with laps of 2 ft.

"A ¾-in. drift bolt 2 ft. long was driven horizontally through both logs at each lap, and in addition the logs were bound together by several turns of wire strand at the pin points. The logs were secured to the fascines by two wrappings of ¼-in. strand, and to every alternate log two of the longitudinal wire strands were made fast.

"Spanning these alternate logs 2-in. manilla ropes were made fast at the same points receiving the longitudinal strands. Each of these rope loops formed an equilateral triangle, with its log for base and with

the shackles of its mattress mooring cables at the vertex. Parallel lines of poles 16 ft. apart were wired to the mattress normal to the current and bank by lashings 5 ft. apart, each alternate lashing being silicon bronze wire. The object of these poles was two-fold; first to give the mattress some stiffness, as without them it was feared the mat would be so flexible as to fold up in sinking; and second, to give to the mattress an element of permanence by its non-corrosive wire lashings. Parallel rows of poles were also placed longitudinally to keep the rock from rolling off."

During the season of 1893 at New Madrid the first large fascine mattress was made. The work consisted of a continuous mattress 250 ft. wide by 900 ft. long, an auxiliary connecting mattress, and a shore paving of 4 ins. of spalls, and 6 ins. of stone extending up the graded bank to the 27-ft. stage. The cost of this work per lineal foot was as follows: River mat fascine, $15 62; connecting mattress fascine, $2 32; paving fascine, $7 58; superintendence and care of plant, $2 26; total, $27 78.

Plate V, Fig. 2, shows a fascine mattress completed. Six mattresses of the fascine type were built and sunk, 658, 825, 913, 1 095, 1 119 and 1 125 ft. long, each being 300 ft. wide. Their cost was $6.282 per square, or $18.846 per lineal foot. Connecting fascine mats were used, costing $11 85 per square. The bank was graded to a 3 to 1 slope after the mattresses were built and sunk, at a cost of 3.8 cents per cubic yard. The shore paving was carried up the bank from the 15-ft. to the 20-ft. stage. In paving, first a layer of spalls 4 ins. thick was laid, and on this rock, 8 ins. at the water edge diminishing to 4 ins. at top. The cost was $10 11 per square, or $5 59 per lineal foot.

The average lineal feet per day's labor in building fascine mats at this point was from 0.386 to 0.495.

Recent Work.—At Hopefield Bend the repairs made in 1890, 1891, 1892 and 1893, constituted practically a renewal of all the original revetment, except two reaches, one 2 000 ft., the other 500 ft. long. These two blocks being destroyed in the flood of 1893, they were renewed by four fascine mattresses 310 ft. wide and 650, 700, 800 and 916 ft. long, costing $6 034 per square, or $18 10 per lineal foot. Connecting mats cost $6.987 per square. The grading cost 5.4 cents per cubic yard. The paving reached from the edge of the connecting mat to 26 to 30 ft. above low water, being carried to this height in order to reach above a sand stratum. The lower 3½ ins. were composed of crushed rock, overlaid

by 6½ ins. of ordinary riprap. The cost was $9 20 per square. The total cost per lineal foot of bank protected was $31 52. For distribution of material see Appendix A.

In building the four mattresses the following averages were made in a day of eight hours.

Mattress.	Length.	Maximum feet made in one day.	Average feet made in one day.	Average number men worked per day.
No. 1.....	650	140	93	220
No. 2.....	800	135	127	280
No. 3.....	700	175	140	300
No. 4.....	916	173	153	320

At Bolivar Front the upper-bank brush revetment, having rotted in many places, was renewed by a paving of stone 10 ins. thick.

Soundings along this revetted reach showed that the inner portion of the old mattresses had silted up, but at their outer edge they had settled down from 5 to 15 ft. At the lower end of the reach during the season woven mattresses 300 ft. wide were constructed and sunk. They were nearly 2 ft. thick at the inner edge, being reduced to 1 ft. in thickness at outer edge. A much greater amount of brush was used in their construction than had before been used. For the distribution of material in them see Appendix A.

The cost was $25 08 per lineal foot, and $8 36 per square. Similar work at Delta Point cost $26.321 per lineal foot.

In New Orleans Harbor up to 1894 the following work had been done:

A continuous mat 400 ft. long placed in Carrollton Bend; five spur dikes, with intermediate revetment covering three upper intervals and about one-third the lower interval in Carrollton Bend; two spur dikes in Greenville Bend; six spur dikes in Gouldsboro Bend; eight spur dikes in Third District Reach.

In the Third District and Carrollton Bends caving took place in the intervals between the dikes. The officer in charge states:

"Spur dikes without intermediate revetment have been successful in some straight reaches, and on concave banks of large radii, but in the abrupt bends the dikes alone are only locally effective."

In the season 1893-94, woven mattresses were still being constructed and sunk in the Third District, at Ashbrook Neck, Greenville Harbor and Lake Providence. At the latter locality fascine work was also

undertaken and very satisfactory results obtained. The mattresses at the other three points specified were reinforced by an extra amount of brush, at Ashbrook Neck the inner 100 ft. only having an additional layer of brush added, while at Greenville Harbor and Lake Providence Reach the extra layer was added all over the mat. The engineer in charge justly condemned the practice of obtaining additional thickness in woven mattresses by the extra layer of brush, on account of the increased stiffness attained. This was the general verdict at all other points, and the necessity for a remedy for this fault produced the fascine type. The fascine mats used at Lake Providence during this season were similar to those of the upper districts, being sunk with fascines normal to the bank.

At Lake Providence the stone on the subaqueous work was increased to 1¼ cu. yds. per square, enough to cover the mattress 4 ins. deep, and the same amount deducted from the upper-bank paving, it being deemed better practice than the old method of 10 ins. of paving and a light mat covering. The engineer recommended spalls on the upper bank in preference to larger riprap, as it packs closer, leaving fewer spaces for the water to penetrate.

The following is a statement giving the cost per unit of labor and comparative average cost per lineal foot of revetment work at Ashbrook Neck, Greenville Harbor and Lake Providence during the season 1994–95.

Cost per Unit of Labor.

Kind of work.	Ashbrook Neck.	Greenville.	Lake Providence.
Mat work, woven, per square	$1.249	$1.290	$1.824
Mat work, fascine, per square	.....	.....	2.171
Grading, hydraulic, per 100 lin. ft	1.736	2.308	1.228
Grading, hand, per 100 lin. ft	.....	3.921	0.629
Paving slope, per square	1.682	1.102	0.863

For the distribution of material on these works see Appendix A.

Comparative Cost and Percentage of Labor and Material.

Items.	Ashbrook Neck. Per cent.	Greenville. Per cent.	Lake Providence. Per cent.
Labor	45.3	36.3	46.6
Material	54.7	63.7	53.4
Cost per lineal foot	$30 25	$28 22	$30 41

The average cost of all this work during the season was $29 33, that during the season of 1892–93 being $28, an increase of $1 33 in 1894. About 50% more brush and 15% more stone were used, which would more than account for the increased price.

Conclusion.—Since the season of 1894 there has been little change in the general form of bank revetment.

The vertical space is left at the top of the bank. The rest of the upper slope is paved, and the subaqueous work is of the fascine type, both continuous and auxiliary. The cost of protection per lineal foot has been approximately $30 at most troublesome localities.

Improvements, both of a mechanical and economical nature, have been attained. The fascine formers have been placed on the mattress barge, thus dispensing with intermediate barges, and mechanical devices have been designed to save labor and perform the work with greater facility and better results.

It will be noticed that in the transverse fascine mat, by dispensing with the ties which form the fascines, that is, by placing the willows in the fascine mattress, when made of the strongest form, with top as well as bottom continuous longitudinal cables, there will result a mattress somewhat similar to the old type of wire net construction used in 1880, but of much greater dimensions and infinitely stronger.

The later mattresses used in New Orleans resemble very much the old wooden pin form used at Delta Point and Memphis prior to 1880, though they are also much stronger.

The early types were more compact than the woven mattress which was adopted in 1882 and used up to 1895, and though the latter was increased in size and given a strength adequate to resist all strains, it was impossible to weave it sufficiently close and compact to prevent the erosive action of the water on the bank through the openings between the willows.

All the recent surveys of the revetted reaches tend to prove that where the bank is protected, no matter how strong the revetment, the channel is deepened just outside of the subaqueous work, and under its outside edge, causing it to take a steep grade. If the mattress is built with sufficient flexibility, strength and compactness, the ultimate result will be the steepening of the subaqueous slope without destroying the efficiency of the work.

When submerged spur dikes are placed in the caving bends, their

efficiency is dependent, to a great extent, on the radius of the bend, the distance between them, and the material of which the bank is composed.

Where the bend is very abrupt and the bank sandy, dikes are of little value. Where the bend is abrupt and the material clay and buckshot, they should be placed not farther apart than 500 ft., and even then it may be necessary to protect the intervals between them.

A thorough knowledge of the river in the vicinity of a reach to be protected is of great importance. Much money can be wasted by not studying the movement of its currents and bars in the locality, in order to select the best point for beginning the work. If placed too high under the bar, dead water may soon prove it a waste of material and an unnecessary expense, while at localities where the bar is receding the failure to place the upper end of the work at the correct place may prove disastrous. A careful survey of the river in the vicinity is very essential.

In the hydraulic grading and in paving the upper bank, the greatest trouble lies in the difficulty with which uniform slopes are obtained in certain materials and with strata in certain relative positions. It is often necessary to do considerable dressing and regrading with sluice boxes, shovels or teams, where the sand strata become washed in pockets some distance below the required grade.

Appendix B gives the distribution of material in the work, and the cost from 1878 to 1894.

The conditions under which the revetment was constructed, the variation in price of material and labor, the difference in form of construction and the irregular manner in which the records have been kept make any comparison of the different forms of doubtful value.

As the education of the labor to the methods has become more nearly perfect, the cost of the labor item per unit of product has decreased; but the great increase in strength, extra use of iron, etc., has partly compensated for that decrease. Since 1878 the cost of rock, brush and poles has decreased, also that of other articles used. The rate of laborer's pay per hour has remained practically the same from 1882 to the present time. The cost of subsistence has not varied to a great extent, notwithstanding the difference in cost of food supplies at different times.

The bank revetment work which the author has endeavored to de-

scribe is probably more extensive than any like engineering construction in the world. A mattress 300 ft. wide by 1 200 ft. long represents a superficial area of about 8 acres, and when one realizes that this vast willow carpet, over a foot thick, is placed on the bottom of the river in depths of from 40 to 100 ft., and against currents of from 5 to 8 ft. per second, the difficulty of the enterprise will be appreciated.

Though much of the revetment from Cairo to New Orleans has needed repairs from year to year, and in some reaches has required renewal as a whole, it may be said to have been eminently successful in the protection of harbor fronts and the prevention of cut-offs and outlets, and fairly so in the control of bank caving, and the resulting change in position and flow of the river.

At some points, where the material of the bank was friable and the currents very strong, the earlier forms of revetment proved too light and were entirely swept away, the shore line continuing to move back. Also considerable reaches of protection work needing repairs and reinforcement at the ends have been destroyed because of the lack of funds, due to the failure of appropriations, etc. In the later work the results have been beneficial and satisfactory, and the loss but slight.

APPENDIX A.

The following tables give the distribution of material used and the cost of revetment per unit, from 1878 to 1894. A square is equal to 100 sq. ft. Except where hardwood is designated, willow was used for brush, and willow or cottonwood for poles. Except where spalls are specified, the rock was in pieces varying from 10 to 100 lbs. in weight.

MEMPHIS HARBOR, 1878 TO 1880, MATS AND REVETMENT; 2 385 SQUARES OR 1 300 LIN. FT.

Materials.	Total amount.	Per square.	Per lineal foot.
Willow brush, cords	2 760	1.16	2.12
Cottonwood poles, number	7 520	3.15	5.80
White oak pins, 4 ft. x 1 in.	800	.33	.62
" 12 x $\frac{3}{4}$ ins	6 300	2.64	4.85
No. 12 wire, pounds	1 950	.82	1.50
Stone, cubic yards	1 154	.48	.88
Sandbags, number	1 500	.63	1.15
Cypress piles, 40 ft. x $1\frac{1}{2}$ ins.	50	one every 26 ft.	

MEMPHIS, 1881.

This work embraced 900 squares of mattress and 131 squares of upper-bank revetment. The distribution of materials was as follows: Willow brush, 1 546 cords, 1.5 cords per square; cottonwood poles, 10 cords, 0.01 cord per square; wire, 2 430 lbs., 2.3 lbs. per square; gravel, 570 cu. yds., 0.55 cu. yd. per square; stone, 187 cu. yds., 0.18 cu. yd. per square; sacks, 1 750, 1.7 per square.

WOVEN MATTRESS WORK, DELTA POINT, 1882 AND 1883.

	Amount.		Cost.	
Items.	Per square.	Per lineal foot.	Per square.	Per lineal foot.
Brush, cords	.41	.6	$1 03	$1 52
Poles, number	1.2	1.66	38	53
No. 12 wire, pounds	1.2	1.70	06	08
Spikes, 6-in., pounds	.34	.5	02	03
Nails, pounds	.21	.3	01	01
Rock, cubic yards	.42	.6	80	1 14
Labor	....	....	60	86

One man built 28 squares, or 2 lin. ft. per day. The cost of the mattress was $2 90 per square, or $4 17 per lineal foot.

UPPER-BANK REVETMENT WORK, DELTA POINT, 1881 TO 1883.

	Amount.		Cost.	
Items.	Per square.	Per lineal foot.	Per square.	Per lineal foot.
Brush, cords	.4	.3	$1 01	$0 76
Poles, number	2.0	1 5	64	48
No. 12 wire, pounds	5.0	3.75	25	19
Spikes, 6-in., pounds	.8	.6	04	03
Rock, cubic yards	2.0	1.5	3 80	2 85
Labor	...	....	1 15	86

One man built $1\frac{1}{2}$ squares or 2 lin. ft. per day. The cost per square was $6 89, and the cost per lineal foot was $5 17. The mat and upper-bank revetment cost about $4 05 per square or $9 34 per lineal foot.

Mats and Revetment, Memphis, 1882 and 1883; 2 005 Squares.

Materials per square: Willow brush, 1.4 cords; cottonwood poles, 2.5 cords; oak pins, 4 ft. by 1 in., 0.35; oak pins, 1 ft. by ¾ in., 1; wire, 1.4 lbs.; stone, 0.54 cu. yds.; sand, 0.1 bag.

Hopefield Bend, 1882 and 1883.

The following statement gives the distribution of material in 1 127 lin. ft. of mattress 140 ft. wide, 194 x 25 ft. of foot mat, and 194 x 16 ft. of upper-bank revetment; 1 657 squares in all. Willow brush, 840 cords, 0.5 cord per square; poles, 105 cords, 0.1 cord per square; wire, 2 534 lbs., 1.5 lbs. per square; rope, 80 lbs., 0.05 lbs. per square; spikes, wrought, 1 950 lbs., 1.2 lbs. per square; nails, 200 lb., 0.12 lb. per square; stone, 585 cu. yds., 0.35 cu. yd. per square; gunny bags, each containing 120 lbs. of clay, 4 300, 2.6 per square.

Lake Providence, 1882 and 1883; Mats and Brush Revetment, 250 Squares or 100 Lin. Ft.

Items.	Amount. Per square.	Amount. Per lineal foot.	Cost. Per square.	Cost. Per lineal foot.
Brush, cords	.61	1.54	$1 07	$2 69
Stone, cubic yards	.41	1.02	82	2 04
Poles, cords	.05	.12	10	25
Spikes, pounds	.51	1.27	02	06
Wire, pounds	.89	2.23	06	16
Iron rods, pounds	4.00	10.00	20	50
Towing	....	.....	33	83
Labor	....	.....	1 87	4 67
Total cost			$4 47	$11 20

The mats were 250 ft. wide; the foot mats, 40 ft.; the revetment, 60 ft.

Plum Point Reach, 1884.

Mat, 175 x 2 010 ft.; 3 517 squares.

Materials.	Total amount.	Per square.	Per lineal foot.
Brush, cords	1 741.41	0.49	0.86
Poles, cords	198.70	0.05	0.09
Wire, pounds	9 175	2.61	4.56

Mat, 175 x 1 750 ft.; 3 062 squares.

Materials.	Total amount.	Per square.	Per lineal foot.
Brush, cords	1 417.75	0.46	0.81
Poles, cords	149.5	0.05	0.08
Wire, pounds	8 650.0	2.82	4.94
Stone, cubic yards	1 757.5	0.57	1.00

Mat, 175 x 1 713 ft.; 2 998 squares.

Materials.	Total amount.	Per square.	Per lineal foot.
Brush, cords	1 982	0.66	1.15
Poles, cords	210	0.07	0.12
Wire, pounds	11 100	3.70	6.48
Stone, cubic yards	1 696	0.57	0.99

Memphis Harbor, 1884 and 1885.

The following amounts are for upper-bank and subaqueous work combined, a total of 12 937 squares. The mats were 150 to 250 ft. wide and 1 500 ft. long. Brush, 11 978 cords, 0.93 cord per square; poles, 440 cords, 0.03 cord per square; stone, 13 654 cu. yds., 1.06 cu. yds. per square; wire, 95 142 lbs., 7.36 lbs. per square; wire rope, 14 733 lbs., 1.14 lbs. per square; iron rods, 85 867 lbs., 6.64 lbs. per square; spikes, 22 500 lbs., 1.74 lbs. per square.

Hopefield Bend, 1884 and 1885.

The following materials were used in constructing 11 003 squares of upper-bank and subaqueous work combined, the mattresses being 150 ft. wide and 2 978 ft. long: Brush, 14 674 cords, 1.33 cords per square; poles, 282 cords, 0.02 cord per square; stone, 6 487 cu. yds., 0.59 cu. yd. per square; wire, 98 090 lbs., 8.91 lbs. per square; spikes, 17 800 lbs., 1.62 lbs. per square; gravel, 3 135 cu. yds., 0.29 cu. yd. per square.

Hopefield Bend, 1885 and 1886; 2 813 Squares, Continuous and Connecting Mattress.

Items.	Amount.	Cost.	Material per square.	Cost per square.
Pay-roll	....	$2 539 92	....	$0 90
Subsistence	....	949 28	....	34
Brush, cords	1 818	2 255 95	.64	80
Poles, cords	171½	291 17	.06	10
Stone, cubic yards	1 620	1 620 00	.58	57
Iron, pounds	8 640	393 76	3.07	14
Wire, pounds	24 310	924 51	8.64	33
Wire rope, feet	4 825	205 06	1.72	08
Spikes and nails, pounds.	5 150	165 20	1.83	06
Tug hire	....	35 00	....	01
		$9 379 85		$3 33

The woven mats were 150 ft. wide. The cost of bank covering was $3 85 per square.

Hopefield Bend, 1885 and 1886; 800 Squares Upper-Bank Revetment and Grading.

Items.	Amount.	Cost.	Material per square.	Cost per square.
Pay-roll	....	$1 994 28	....	$2 49
Subsistence	....	639 00	....	80
Brush, cords	1 114	1 371 55	1.39	1 71
Poles, cords	80	135 83	.10	17
Stone, cubic yards	882	882 00	1.10	1 10
Iron, pounds	1 822	57 43	2.28	07
Wire, pounds	11 260	435 42	14.07	55
Spikes and nails, pounds	2 350	64 80	2.94	08
Coal, bushels	892½	89 25	1.11	00
Miscellaneous	....	1 75	....	11
		$5 671 31		$7 08

This does not include the cost of general repairs, administration, etc. The area was 1 140 ft. long and 70 ft. wide.

MEMPHIS DIKE CONSTRUCTION, 1886 AND 1887.

Subaqueous or Foundation Mattress, 3 524 squares.

Items.	Amount.	Cost.	Amount per square.	Cost. per square.
Labor	.....	$4 196 27	....	$1 19
Subsistence	.....	.....	....	....
Brush, cords	2 208.6	2 164 43	.63	62
Poles, cords	465.5	632 17	.13	18
Stone, cubic yards	1 997.7	2 188 44	.57	62
Spikes, pounds	4 000	104 00	1.13	03
Iron, pounds	1 035	18 98	.30	01
Lumber, feet	6 884	94 28	1.95	02
Manilla rope, pounds	4 154	465 32	1.18	13
Miscellaneous	.....	110 78	....	03
Wire, pounds	18 500	740 00	5.22	21
Wire cable, pounds	10 500	459 37	2.98	13
		$11 174 04		$3 17

MEMPHIS DIKE CONSTRUCTION, 1886 AND 1887.

1 112 Squares of Upper Bank Revetment.

Items.	Amount.	Cost.	Amount per square.	Cost per square.
Labor	....	$2 239 58	....	$2 02
Subsistence	....	....	....	....
Brush, cords	1 550	1 667 68	1.39	50
Poles, cords	145	189 70	.13	17
Stone, cubic yards	1 190	1 304 76	1.07	1 16
Spikes, pounds	2 700	81 00	2.43	07
Wire, pounds	6 704	268 16	6.03	25
		$5 750 88		$5 17

These amounts do not include grading for the shore cribs.

MEMPHIS DIKE CONSTRUCTION, 1886 AND 1887.

1 003 898 Cu. Ft. of Cribwork.

Items.	Amount.	Cost.	Amount per cubic foot.	Cost per cubic foot.
Labor	....	$10 159 56	....	$0.01
Brush, cords	4 003	3 933 20	0.004	.004
Poles, cords	987	1 292 11	.0009	.001
Stone, cubic yards	4 494	4 929 54	.005	.005
Spikes, pounds	13 553	383 89	.01	.0004
Miscellaneous	....	155 74	....	.0002
Wire, pounds	42 824	1 712 96	.04	.002
Wire cable, pounds	25 675	1 123 28	.02	.001
Iron, pounds	7 260	153 11	.007	.0001
		$23 843 39		$0.024

MEMPHIS, 1886 AND 1887, SUBAQUEOUS MATTRESS; 7 856 SQUARES OR 2 950 LIN. FT.

Items.	Amount.	Cost.	Amount per square.	Cost per square.
Labor	....	$8 358 03	....	$1 06
Subsistence	...	400 33	....	05
Brush, cords	5 054	5 620 04	0.64	72
Poles, cords	765	1 197 50	.09	15
Stone, cubic yards	4 195	7 073 61	.53	90
Wire, pounds	38 498	1 539 92	4.90	20
Wire cable, pounds	15 693	697 98	2.00	08
Spikes and nails, pounds	7 700	207 45	.98	03
Iron, pounds	5 477	148 01	.70	02
Lumber, feet	9 044	150 78	1.15	02
Manilla rope, pounds	2 360	264 50	.30	03
Coal, boxes	200	75 00	.02	01
Miscellaneous	....	113 60	....	02
		$25 846 75		$3 29

The cost per lineal foot of this continuous mattress was $8 76. Including repairs to plant, all office and administration expenses, etc., it was $4 52 per square and $11 97 per lineal foot.

HOPEFIELD BEND, 1887 AND 1888, SUBAQUEOUS MATTRESS; 7 546 SQUARES.

Items.	Amount.	Cost.	Amount per square.	Cost per square.
Labor	....	$5 491 95	....	$0 73
Subsistence	....	2 241 80	....	30
Brush, cords	3 752	4 314 80	0.50	57
Poles, cords	833	1 457 75	.11	19
Stone, cubic yards	3 821	5 784 96	.50	76
Wire, pounds	39 708	1 588 32	5.25	21
Wire strand, pounds	11 530	547 67	.153	07
Spikes and nails, pounds	6 900	205 45	.91	03
Lumber, feet	3 100	62 00	.41	01
Iron, pounds	1 130	29 75	.15	003
Manilla rope, pounds	1 200	135 00	.16	01
Miscellaneous	....	24 28	....	003
		$21 883 73		$2 89

HOPEFIELD BEND, 1887 AND 1888, UPPER-BANK REVETMENT, INCLUDING GRADING; 6 182 SQUARES.

Items.	Amount.	Cost.	Amount per square.	Cost per square.
Labor	....	$8 092 72	....	$1 31
Subsistence	....	2 907 74	....	47
Brush, cords	5 470	6 521 94	.88	1 05
Poles, cords	603	1 031 65	.10	17
Stone, cubic yards	4 032	6 103 42	.65	99
Wire, pounds	34 678	1 387 12	5.61	22
Wire strand, pounds	1 450	68 88	.24	01
Spikes and nails, pounds	2 400	73 20	.39	01
Coal, boxes	2 103	787 20	.34	13
Iron, pounds	915	22 77	.15	003
Miscellaneous	....	125 79	....	02
		$27 122 43		$4 38

HOPEFIELD BEND, 1888 AND 1889, SUBAQUEOUS MATTRESS, 196 FT. WIDE; 9 394 SQUARES.

Items.	Amount.	Cost.	Amount per square.	Cost per square.
Labor	....	$8 191 35	....	$0 87
Subsistence	....	2 958 86	....	32
Brush, cords	4 977	6 125 60	0.53	65
Poles, cords	678	1 185 50	.07	13
Stone, tons	4 442	6 973 94	.47	74
Wire, pounds	48 814	1 911 83	5.20	20
Wire strand, pounds	12 620	534 98	1.34	06
Spikes, pounds	3 300	90 81	.35	01
Lumber, feet	6 325	104 61	.67	01
Iron, pounds	1 623	40 57	.17	004
Manilla rope, pounds	1 650	184 80	.18	02
Miscellaneous	....	43 22	....	005
		$28 346 07		$3 02

HOPEFIELD BEND, 1888 AND 1889, UPPER-BANK REVETMENT, ABOUT 142 FT. WIDE; 6 635 SQUARES COVERED.

Items.	Amount.	Cost.	Amount per square.	Cost per square.
Labor	....	$8 572 37	....	$1 30
Subsistence	....	3 095 92	....	47
Brush, cords	6 233	7 885 20	0.94	1 19
Poles, cords	621	1 131 75	.09	17
Stone, tons	5 089	7 989 73	.77	1 20
Wire, pounds	53 095	1 821 38	8.00	27
Wire strand, pounds	336	13 43	.05	002
Spikes, pounds	2 000	58 04	.30	01
Manilla rope, pounds	400	44 80	.06	007
Miscellananeous	....	21 34	....	003
		$30 633 96		$4 62

The above does not include grading.

BOLIVAR, 1888 AND 1889.

This work comprised 10 300 squares of mattress, 180 to 250 ft. wide, and 2 842 squares of upper-bank revetment. The amount of materials was as follows: Brush, 9 639 cords, 0.73 cord per square; poles, 1 659 cords, 0.12 cord per square; stone, 10 154 cu. yds., 0.77 cu. yd. per square; wire, 129 310 lbs., 984 lbs. per square; wire cable, 25 025 lbs., 1.9 lbs. per square; spikes, 33 100 lbs., 2.52 lbs. per square; iron rods, 51 924 lbs., 3.95 lbs. per square; clevises, lap rings, 8 500 lbs., 0.65 lb. per square; lumber, 7 081 ft., 0.54 ft. per square; staples, 1 858 lbs., 0.01 lb. per square.

HICKMAN HARBOR, 1889 AND 1890, 2 736 SQUARES OF SUBAQUEOUS MATTRESS, 300 FT. WIDE.

Items.	Amount.	Cost.	Amount per square.	Cost per square.
Labor	.....	$3 637 99	..	$1 33
Subsistence	.....	104 78	....	04
Brush, cords	1 812	1 902 60	.66	70
Poles, cords	250	312 50	.09	12
Stone, cubic yards	1 717	2 508 14	.62	91
Wire, pounds	22 560	621 83	8.22	22
Wire strand, pounds	4 061	1 45 79	1.48	05
Spikes and nails, pounds	1 000	27 50	.36	01
Lumber, feet	6 985	75 10	2.56	03
Iron, pounds	1 480	20 60	.54	01
Manilla rope, pounds	700	72 55	.25	02
Piling, feet	455	36 40	.17	02
		$9 465 78		$3 46

HICKMAN HARBOR, 1889 AND 1890, 1 378 SQUARES OF UPPER-BANK REVETMENT 143 FT. WIDE, GRADING INCLUDED.

Items.	Amount.	Cost.	Amount per square.	Cost per square.
Labor	.....	$4 571 41	....	$3 31
Subsistence	.....	305 00	....	22
Brush, cords	1 317	1 382 85	0.95	1 00
Poles, cords	131	163 50	.09	1185
Stone, cubic yards	1 508	2 202 90	1.095	1 60
Wire, pounds	5 524	155 10	4.01	1135
Spikes and nails, pounds	300	8 25	.22	006
Lumber, feet	1 000	15 00	.725	011
Coal, bushels	2 292	229 20	1.662	166
Oil, etc	.....	76 58	....	055
		$9 109 79		$6.000

The total revetment on this work, above and below water cost, including charges on plant, etc., $6 00 per square and $24 77 per lineal foot.

LOUISIANA BEND, 1889.

This work comprised 21 695 squares of subaqueous mattress and 5 672 squares of paving, 10 ins. thick. The materials used were as follows: Stone, 48 161 cu. yds., 1.76 cu. yds. per square; brush, 18 813 cords, 0.68 cord per square; poles, 3 362 cords, 0.12 cord per square; wire, 167 440 lbs., 6.16 lbs. per square; wire cable, 329 560 lbs., 12.02 lbs. per square; spikes, 36 961 lbs., 1.35 lbs. per square.

GREENVILLE, 1889; DIKE NO. 2.

Foundation Mattress, 290 x 290 Ft. or 841 Squares.

Materials.	Amount.	Cost per unit.	Cost, total.	Amount per square.	Cost per square.
Brush, cords	553	$1 27½	$705 07	0.657	$0 84
Poles, cords	141	1 47½	207 97	.168	25
Stone, tons	767	1 95	1 496 65	.912	1 78
Wire, coils	34	3 37	114 58	.040	14
Cable, coils	6	42 27	253 62	.007	30
Spikes, 6-in., kegs	4	3 25	13 00	...	..
Spikes, 9-in., kegs	5½	3 12½	15 63	.011	03
Staples, kegs	1¼	5 00	6 25	.001	01
Total materials			$2 811 77		$3 35
Labor, hours, including subsistence	5 630	16	$900 80	6.69	$1 07
Sailor work, hours, including subsistence	737	16	117 92	0.88	14
Total labor			$1 018 72		$1 21
Grand total			$3 830 49		$4 56

Crib No. 1, 212 32 x 8 Ft.; 54 272 Cu. Ft.

Materials.	Amount.	Cost per unit.	Cost, total.	Amount per 100 cubic feet.	Cost per 100 cubic feet.
Brush, cords	228	$1 27½	$290 70	0.420	$0 54
Poles, cords	100	1 48	148 00	.184	27
Stone, tons	324	1 50	486 00	.597	90
Wire, coils	32	3 37	107 84	.059	20
Cable, coils	2½	42 27	105 67	.005	20
Spikes, 6-in., kegs	2¾	3 25	8 94	....	..
Spikes, 9-in., kegs	7¾	3 12½	24 22	.013	05
Total material cost.			$1 171 37		$2 16
Labor, hours, including subsistence	2 686	16	$429 76	4.95	$0 79
Sailor work, hours, including subsistence.	610	16	97 60	1.12	18
Total			$527 36		$0 97
Grand total			$1 698 73		$3 13

Crib No. 2, 170 x 16 x 8 Ft.; 21 760 Cu. Ft.

Brush, cords	95	$1 27½	$121 12	.437	$0 55
Poles, cords	52	1 48	76 96	.239	35
Stone, tons	119	1 95	232 05	.547	1 08
Wire, coils	27	3 37	90 99	.124	42
Cable, coils	1½	42 27	63 40	.007	29
Spikes, 6-in., kegs	1¾	3 25	5 69	....	..
Spikes, 9-in., kegs	2¼	3 12½	7 03	....	..
Spikes, 9-in., kegs	1½	5 70	8 55	.002	09
Total			$605 79		$2 78
Labor, including subsistence, hours	1 351	16	$216 16	6.21	$0 99
Sailor work, including subsistence, hours	310	16	49 60	1.42	23
Total			$265 76		$1 22
Grand total			$871 55		$4 00

Revetment, 230 Squares.

Materials.	Amount.	Cost per unit.	Total cost.	Amount per square.	Cost per square.
Brush, cords	325	$1 09	$354 25	1.413	$1 54
Poles, cords	45	1 48	66 60	.196	29
Stone, tons	649	1 95	1 265 55	2.822	5 50
Wire, coils	22	3 37	74 14	.096	32
Spikes, 6-in., kegs	½	3 25	1 62	....	..
Spikes, 9-in., kegs	3	3 12½	9 37	....	..
Spikes, 9-in., kegs	½	5 70	2 85	.017	06
Total material			$1 774 38		$7 71
Grading, labor and subsistence, hours	5 000	16	$800 00	21.74	$3 48
Construction, labor and subsistence, hours	3 584	16	593 44	15.58	2 58
Total			$1 393 44		$6 08
Grand total			$3 167 82		$13 77

Land Crib, 3 000 cu. ft.

	Amount.	Cost per unit.	Total cost.	Amount per 100 cubic feet.	Cost per 100 cubic feet.
Brush, cords	7	$1 09	$7 63	0.233	$0 25
Poles, cords	10	1 48	14 80	.333	49
Stone, cords	16	1 95	31 20	.533	1 04
Wire, coils	2	3 37	6 74	.066	22
Total materials			$60 37		$2 00
Labor, including subsistence, hours		16	32 32	6.073	1 08
Grand total			$92 69		$3 08

New Orleans Harbor, 1890 and 1891.

Foundation Mat, 355 x 130 x 1.75 Ft., or 461 Squares.

Items.	Amount.	Cost.	Amount per square.	Cost per square.
Brush, cords	608	$1 258 35	1.32	$2 73
Lumber, feet	16 222	176 80	3.51	38
Rock, tons	299	597 34	.65	1 30
Wire, pounds	991	39 64	2.15	08
Nails, pounds	4 200	132 90	9.12	29
Chain, pounds	3 787	132 54	8.240	28
Bolts, number	86	4 30	·18	01
Fish plates, number	131	9 17	.28	02
Subsistence		360 00	....	78
Labor		1 995 91	....	4 33
Total		$4 706 95		$10 20

Cribs, 2 325 Cu. Ft.

Brush, 1 387 cords, costing $2 870 67; poles, 68 cords, costing $140 76; rock, 580 tons, costing $1 160 09; lumber, 37 380 ft., costing $407 44; No. 10 wire, 2 090 lbs., costing $83 60; iron rods, 456 lbs., costing $15 96; nails, 2 150 lbs., costing $74 25; subsistence, $450; labor, $3 100 16. Total cost, $8 303 74; cost per cubic foot, 3.57 cents.

Ashbrook Neck, 1891 and 1892, Mattresses 300 Ft. Wide and Upper-Bank Paving.

This work required the following materials: Brush and poles, 2.565 cords per lineal foot; stone, 5.136 cu. yds.; wire cables, 14.8 lbs.; wire, 19.517 lbs.; spikes, 2.6 lbs. The distribution per square was as follows: Brush, 0.661 cord; poles, 0.132 cord; stone, 0.688 cu. yd. The cost per unit is given in the tabular statement immediately following.

	Labor and subsistence.	Material.	Total.
Mat work, per square	$1.573	$3.117	$4.690
Foot mat, per square	1.719	3.616	5.333
Revetment, per square	2.388	6.809	9.197
Paving, per square	1.617	7.006	8.623
Clearing bank, per acre	71.500	.....	71.50
Loading stone, per cubic yard	0.589	.....	0.589
Grading, per lineal foot	0.840	0.369	1.210
Dressing, per lineal foot	0.457	.....	0.457

GREENVILLE, 1891 AND 1892; 300-FT. WOVEN MAT AND PAVING.

Kind of work.	Cost per Unit. Labor and subsistence.	Material.	Total.
Mattress, per square	$1.332	$3.167	$4.499
Revetment, per square	2.042	7.502	9.544
Paving, per square	1.121	7.474	8.594
Grading, per lineal foot	.5387	.213	.7523
Dressing, per lineal foot	.0755	.00	.6755
Loading stone, per cubic yard	.4520	.003	.4550

LOUISIANA BEND, 1891 AND 1892, 270-FT. MATS AND 10-IN. PAVING.

Materials.	Mattress.	Connecting mat.	Upper-bank revetment.	Paving.
Brush, cords	0.8	0.8	1.2	
Poles, cords	.12	.12	.12	
Stone, cubic yards	.7	1.25	1.25	3.00
Wire, pounds	5.2	5.2	5.2	
Cable, pounds	4.4	4.4	4.4	
Spikes, pounds	0.16	0.16		

PLUM POINT REACH, 1892 AND 1893.

River Mattress, Heaviest Woven Type.—Brush, 0.948 cord per square; poles, 0.111 cord; stone, 0.647 cu. yd.; wire, 8.07 lbs.; wire strand, 2.9 lbs.; spikes, 0.4 lb.; cable clamps, 0.114; staples, 0.048 lb.; piles, 0.0092.

Paving.—Stone, 0.385 cu. yd. per square, and 1.709 cu. yds. per lineal foot; spalls, 0.113 and 0.501 cu. yd.

Connecting Mats.—Brush, 1.117 cords per square; poles, 0.136 cord; stone, 2.228 cu. yds.; wire, 6.716 lbs.; wire strand, 2.024 lbs.; spikes, 0.24 lb.; cable clamps, 0.0078; staples, 0.0012 lb.

Pocket Mats.—Brush, 0.83 cord per square; poles, 0.145 cord; stone, 1.55 cu. yds.; wire, 7.44 lbs.; wire strand, 2 lbs.; clamps, 0.065; staples, 0.025 lb.; spikes, 0.206 lb.

Cost of Work.—River mats, $4.27 per square; connecting mats, $8.17 per square; pocket mats, $5.90 per square; paving, $10.11 per square; grading, 3¾ cents per cubic yard; clearing, $42.56 per acre. Cost per lineal foot of revetment complete, $19.22.

ASHBROOK NECK, 1892 AND 1893.

The mattress in this work was 250 ft. wide, and the slope of the paved bank was 4 to 1. The distribution of the materials was as follows: Brush, 0.62 cord per square, 2.025 cords per lineal foot; poles, 0.14 cord per square, and 0.41 cord per lineal foot; stone, 6.137 cu. yds. per lineal foot; wire, 7.7 lbs. per square, and 22.95 lbs. per lineal foot; wire cable, 4 and 13.6 lbs. This work cost $29 07 per lineal foot.

GREENVILLE, 1892 AND 1893.

This work included woven mattresses and a paved upper bank on a slope of 4 to 1. The distribution of materials was as follows: Brush, 0.71 cord per square and 2.55 cords per lineal foot; poles, 0.13 and 0.35 cord; stone for mattress, 0.63 cu. yd. per square; stone for paving, 3.03 cu. yds. per square; total stone, 5.74 cu. yds. per lineal foot; wire, 5.36 lbs. per square, and 20.98 lbs. per lineal foot; wire cable, 4.28 and 14.92 lbs. The work cost $27 08 per lineal foot.

Louisiana Bend, 1892 and 1893.

The work comprised woven mats and upper-bank paving. The distribution of materials was as follows : Brush, 0.88 cord per square and 2.63 cords per lineal foot ; poles, 0.17 and 0.44 cord ; stone for mat, 0.63 cu. yd. per square ; stone for paving, 3.58 cu. yds. per square ; total stone, 6.25 cu. yds. per lineal foot ; wire, 5.61 lbs. per square and 20.14 lbs. per lineal foot ; wire cable, 3.53 and 10.69 lbs.

Bolivar Front, 1893.

This work was a mat 300 ft. wide, 2 ft. thick at the upper edge and 1 ft. at the lower. The distribution of the materials was as follows : Brush, 0.98 cord per square ; poles, 0.15 cord ; stone, 0.53 cu. yd.; wire, 5.5 lbs.; wire cable, 4.29 lbs.; spikes, 0.75 lb.

Hopefield Bend, 1893 and 1894.

The work included a fascine mattress 310 ft. wide and connecting fascine mattresses. The material in the 310-ft. mattress was distributed as follows : Brush, 1.639 cords per square; poles, 0.053 cord ; stone, 0.625 ton ; steel wire, 4.861 lbs.; copper wire, 0.546 lbs.; wire strand, 10.965 lbs.; clamps, 1.5. The cost per square constructed was $5 94 and the cost per square of bank covered was $6 07. The distribution of the material in the connecting mattresses was as follows : Brush, 2.355 cords per square; poles, 0.122 cord; stone, 0.66 ton. The cost was $6.987 per square.

Distribution of Material, 1894.

Material.	Ashbrook Neck.	Greenville.	Lake Providence.
Brush per square, woven mat, cords	1.163	1.157	0.98
Brush per square, fascine mat, cords	....	....	1.37
Poles per square, woven mat, cords	0.214	0.200	0.14
Stone per lineal foot, cubic yards	6.76	6.66	7.3
Wire per lineal foot, pounds	28.35	30.63	25.38
Wire strand per lineal foot, pounds	15.03	16.00	25.54
Spikes per lineal foot, pounds	4.47	3.20	1.89

APPENDIX B.—Cost of Bank Protection Work, Per Unit, from 1878 to 1895.

Date.	Locality.	Mattress work. Per square.	Connecting mat. Per square.	Upper-Bank Revetment, Including Grading.		Crib work. Per cubic foot.	Mat and upper-bank work combined. Per square.	Complete work. Per lineal foot.	Remarks.
				Brush work. Per square.	Stone paving. Per square.				
1878	New Orleans Harbor.....	$12 87	.....	.....	.....	.....	.....	.....	Cane mats, 200′ x 24′.
1879	" " "	9 88	.....	.....	.....	.....	.....	$13 38	" " "
1879	Delta Point, La.....	12 00	.....	.....	.....	.....	.....	18 00	Pin mats, 150′ x 50′.
1880	New Orleans Harbor.....	7 22	.....	.....	.....	.....	.....	.....	Cane mats, 200′ x 70′.
1882	Delta Point, La.....	2 90	.....	$6 89	.....	.....	.....	14 00	Woven mats, 140′ x 400′.
1883	Plum Point Reach.....	6 30	.....	6 40	.....	.....	.....	.....	Woven mats and brush revetment.
1882-1883	Lake Providence Reach..	.....	.....	.....	.....	.....	$4 47	11 20	Woven mats and brush revetment.
1884	New Orleans Harbor.....	7 60	.....	.....	.....	$0 03⅓	.....	.....	Woven mats, 200′ x 350′, framed cribs.
1885-1886	Hopefield Bend.....	3 85	.....	7 08	.....	.....	.....	.....	Woven mats, 150′ wide, brush revetment.
1886-1887	Memphis Harbor.....	3 17	.....	5 17	.....	.024	.....	.....	Woven mats, frame cribs, including grading.
1886-1887	" "	3 29	.....	.....	.....	.....	.....	.....	Woven mats, continuous revetment.
1887-1888	Hopefield Bend.....	2 89	.....	4 38	.....	.....	.....	.....	Woven mats, continuous revetment.
1888	Greenville Harbor.....	.....	.....	.....	.....	.....	.....	20 20	Dike construction.
	New Orleans Harbor.....	7 54 and 9 11	.....	.....	.....	.0457	.....	.....	Woven mats and cribs.
	Hopefield Bend.....	.....	.....	.....	.....	.....	4 75	.....	Woven mats, brush revetment.
	Bolivar Front.....	.....	.....	.....	.....	.....	9 26	28 31	Woven mats, brush revetment.
1888-1889	Hopefield Bend.....	3 02	.....	4 62	.....	.....	.....	.....	Woven mats, brush revetment.
1889	New Orleans Harbor.....	6 39	.....	.....	.....	.....	.....	.....	Woven mats.
1889	" " "	9 56	.....	.....	.....	.04	.....	.....	Frame mats, frame cribs.

APPENDIX B.—COST OF BANK PROTECTION WORK, PER UNIT, FROM 1878 TO 1895—(*Continued*).

Date.	Locality.	Mattress work. Per square.	Connecting mat. Per square.	Upper-Bank Revetment, Including Grading.		Crib work. Per cubic foot.	Mat and upper-bank work combined. Per square.	Complete work. Per lineal foot.	Remarks.
				Brush work. Per square.	Stone paving. Per square.				
1889-1890	Hickman Harbor	3 46	.....	6.00	.....	.....	.....	24 77	Woven mats, brush revetment.
	Columbus Harbor	.....	.....	.....	.....	.04	6 70	22 65	Woven mats and cribs.
	Helena Harbor	4 89	.....	.....	.....	.044	.....	.....	Woven mats and cribs.
	Greenville Harbor	4 56	.....	7.71	.....	.031 and .04	.....	.....	Woven mats and cribs.
	Bolivar Front	.....	.....	.....	.....	.....	10 68	.....	Woven mats and brush revetment.
1890-1891	New Orleans Harbor	10 20	.....	.....	.....	.0357	.....	.....	Pin mats, frame cribs.
	Ashbrook Neck	.....	.....	.....	.....	.....	7 69	30 42	Woven mats and brush revetment.
	New Orleans Harbor	9 95 and 9 48	.....	.....	.....	.0358 and .0431	.....	.....	Pin mats and framed cribs.
1891-1892	Hopefield Bend	7 02	$3 36	.....	6 12	.....	.....	19 57	Paving 10" stone.
	Ashbrook Neck	4 69	5 33	9.197	8 623	.....	.....	30 83	Woven mat pavement, 10".
	Greenville Harbor	4 50	.....	9.54	8 59	.....	.....	29 52	" " " "
	Louisiana Bend	.....	.....	.....	.....	.....	6 06	26 49	" " " "
	New Orleans Harbor	9 28	.....	.....	.....	.037	.....	.....	Pin mats, frame cribs.
1892-1893	Ashport Bend	4 27	8 17 and 5 90	.....	10 11	.....	.....	19 22	Woven mats and paving.
	Daniels Point	5 09	7 81	.....	.....	.....	.....	.....	" " "
	Hopefield Bend	3 65	5 89	.....	11 70	.....	.....	.....	" " "
	Ashbrook Neck	.....	.....	.....	.....	.....	.....	29 07	" " "
	Greenville Harbor	.....	.....	.....	.....	.....	.....	27 08	" " "
	Louisiana Bend	.....	.....	.....	.....	.....	.....	27 86	" " "
	New Orleans Harbor	8 92	.....	.....	.....	.....	.....	.....	Pin mats.
1893-1894	New Madrid Harbor	.....	.....	.....	.....	.....	.....	27 78	Fascine mats and paving.
	Ashport Bend	6 28	11 85	.....	10 11	.....	.....	.....	" " "
	Hopefield Bend	6 03	6 99	.....	9 20	.....	.....	31 52	" " "
	Bolivar Front	.....	.....	.....	.....	.....	8 36	25 08	Woven mats and paving.
	Delta Point	.....	.....	.....	.....	.....	.....	26 32	" " "
1894-1895	Ashbrook Neck	.....	.....	.....	.....	.....	.....	30 25	" " "
	Greenville Harbor	.....	.....	.....	.....	.....	.....	28 22	" " "
	Lake Providence	.....	.....	.....	.....	.....	.....	30 41	Woven and fascine mats and paving.

In the finished work the percentage of cost of labor and material is about 45 and 55%, respectively.

CORRESPONDENCE.

E. E. Russell Tratman, Assoc. M. Am. Soc. C. E.—The paper deals with the work on the lower Mississippi, but it may be of interest to mention briefly here the work on the upper Mississippi, in Wisconsin and Iowa, carried out in 1894–95 under Alexander Mackenzie, M. Am. Soc. C. E. The bank to be protected was cleared from trees, stumps and snags, and trimmed to a regular slope of one to two. The fascines were 20 ft. long and 12 to 15 ins. diameter, made of well-trimmed live brush closely packed together and tied with bands of wire or tarred rope not more than 3 ft. apart. These were made into mattresses 12 ft. long, a single layer being secured to three pairs of binding poles by ties of wire or tarred rope not more than 2 ft. apart. The mattresses were sunk at the foot of the bank, extending out into the river, and were loaded with a layer of rock 9 to 24 ins. thick, in pieces of 6 to 10 cu. ins., a sufficient amount of small stone or gravel being used to fill all interstices and form a compact mass. The larger stones were also laid to cover the bank above the edge of the mattress. Mr. Tratman.

The construction of brush dikes on the James River, in Virginia, specifications for which were issued in 1889 by William P. Craighill, Past-President Am. Soc. C. E., may also be referred to. These were built in water from 0 to 16 ft. deep at low tide, with a mean rise and fall of tide of $3\frac{1}{3}$ ft. The main dike was approximately parallel with the shore, with lateral dikes connecting it with the shore. The methods of construction were as follows:

First.—Two rows of poles were driven 6 ft. apart, the poles being of pine, 8 ins. diameter at the small end and 35 ft. long (or in some cases 25 ft.), spaced 8 ft. center to center, and sawed off to 7 ft. above high water. Between these and at right angles to the current was placed a layer of live brush to a width of 30 ft. The bottom layer, 3 ft. thick, was of small trees or saplings of sufficient length to be tied together in sections, and projecting 15 ft. from the outside row of poles. This was loaded down with not less than 2 tons of stone or gravel per linear foot of dike. Upon this, and crosswise of the bottom layer, brush was filled in and well packed between the two rows of piles up to the level of the tops of the piles, after which the piles were tied together longitudinally and transversely with galvanized wire of No. 12 American gauge, each set having at least 12 strands of wire, six on each side.

Second.—Similar to the first, but without the bottom layer of brush and stone.

Third.—Where the bottom was so rocky that piles could not be driven, three rows of logs were laid parallel with the current, the rows being 10 ft. apart, made of logs 25 ft. long and 8 ins. diameter at the smaller end. Where the ends of the logs lapped they were bound by

Mr. Tratman. three strands of No. 12 galvanized iron wire. Upon these were laid 8-in. cross logs 3 ft. apart, each fastened to the lower poles by two strands of No. 12 galvanized wire. Upon these, and parallel with the lower logs, brush was piled to a depth of 3 ft., when another layer of cross logs was added, and secured to the cross logs below at every intersection with the bottom logs and half way between the intersections. The 3-ft. layers of brush, separated by rows of cross logs, were carried up until the low-water surface was reached. The dike was then sunk by loading it with stone or gravel, after which it was further consolidated by putting on 2 tons of hard stone per linear foot of dike.

In the regulation of the River Isar, a mountain stream of Bavaria, a system of suspended mattresses has been employed. The heels of the fascines are attached to a row of piling, so as to form a slightly pendulous inclined wall, which checks the current and causes the detritus to be deposited in the comparatively still water behind the fascines, the artificial bank being afterward protected by mattress revetment.

Mattress revetment has been employed for shore protection on Lake Erie, and the following is a description of this work from specifications by Capt. F. A. Mahan for shore protection at Erie, Pa., in 1888. The protection consists of a row of oak piles parallel with and 100 ft. from the shore, 6 ft. center to center, driven 15 ft. into the bed of the lake, and cut off 3 ft. above water. On each side and 12 ins. above the water line is a 6 x 12-in. waling piece, and on the outer side are sheeting piles placed close together, driven 7 ft. into the ground, secured to the waling piece and having another waling piece fastened on the outside. In the rear of this is a brush mattress 20 ft. wide, weighted down by stone. The mattresses are made as follows: Fascines 20 ft. long were made by binding together saplings not less than 13 ft. long or more than 2 ins. diameter at the butt, laid alternately, butts and tips, to form compact bundles 12 ins. diameter, each bundle having six fastenings of No. 10 B. W. G. wire, the extreme ones 6 ins. from the ends. For each mattress there are four poles 29 ft. long and 6 ins. diameter at the smaller end, placed 6 ft. center to center. Across these were laid the fascines, bound by No. 10 wire, passing alternately over two and under one fascine in succession, the wire also passing around the poles. The completed mattresses were towed into place and sunk by loading them with stone, close-grained limestone or granite, in blocks of 100 to 200 lbs. weight.

It would be interesting to know if mattress revetment has been employed for shore protection on lake or sea coast in positions where it is subjected to wave attacks. The writer is under the impression that some such system was employed a few years ago at Brighton Beach, on Coney Island, N. Y., where, however, the Atlantic waves undermined it and broke it to pieces.

WILLIAM STARLING, M. Am. Soc. C. E.—The paper deals with a very important feature of river improvement, which has received an extraordinary development under the Mississippi River Commission. It is not by any means a novel contrivance, for it has been in use in Europe for a century or more. In its application to the peculiar conditions of the Mississippi, it has been necessary to depart more or less widely from the methods in use elsewhere. Many of the devices employed have been original, and evolved from the necessities of the case. The great size and violence of the big American stream and the extraordinary instability of its banks and bed demand works of special strength and dimensions. Brush has to be obtained from the wild growths which spring up spontaneously on the bars and foreshores. It is larger and rougher than the brush of the European markets, which is mostly an artificial product, raised for the special purpose for which it is employed. It is interesting to observe that the latest constructions of bank revetments differ less from the foreign types than those of an earlier date. Mr. Starling.

It is with the details of the work as it is practiced on the Mississippi that the paper deals; and the author has preferred to confine himself to such details rather than to go into the general principles of bank revetment as a branch of river improvement. The author is an authority on the subject of which he writes, as he has been connected with river improvement since its beginnings on the Mississippi. It is well known to those who have kept up with the progress of the latter work that bank protection was not adopted as a principle without violent opposition. The late James B. Eads threw the whole weight of his great name and abilities against it. The friends of revetment believed and still believe that by preventing the caving of banks, they would cut off the supply of material for building up the shoals below, and that thus some of the principal obstacles to navigation would be removed. Mr. Eads did not concur in this view. He thought that if the water was already fully loaded with sediment, it would stop caving, even in concave bends. His remedy for the evil was therefore to contract the wide places above the caving reaches. He was opposed to direct bank protection, first, on account of the enormous expense involved; second, because he thought the work ineffective as a means of improving navigation, and third, because he did not believe that the revetments could be maintained. The matter will be found fully discussed in Mr. Eads' numerous letters, in the paper on caving banks* by B. M. Harrod, M. Am. Soc. C. E., and in other places.

The old inhabitants and the steamboatmen were not only incredulous as to the success of bank protection, but they were sure it could not be accomplished. All the money in the United States Treasury could not do it. The first puny efforts that were made seemed almost

* Published in pamphlet form in 1886.

Mr. Starling. to justify their gloomy forebodings, and it was only by taking to heart the lessons taught by repeated failures that success has at last been attained.

The experience of the last 15 or 16 years has resulted, as the author has noted, in a continuous advance in the direction of strength and expensiveness, until the cost of a thorough system of bank protection in a badly caving reach is about $30 per linear foot. The work thus constructed is not absolutely permanent. Sooner or later, as stated in the paper, there occurs a scour at the foot of the revetment, which causes the latter to assume a steeper slope, and frequently results in a sliding down of parts of the structure, attended with breaks in the fabric, and sometimes with partial erosion in the bank thus left exposed. These damages are not usually extensive, nor is the repair of them generally very costly.

The latest conclusion to which the Mississippi River Commission has come seems to be that bank revetment is a valuable and effective means of preserving the shores of the river from erosion, and in certain situations is the legitimate and necessary remedy to be applied; as, for instance, for the prevention of a cut-off, for the preservation of a

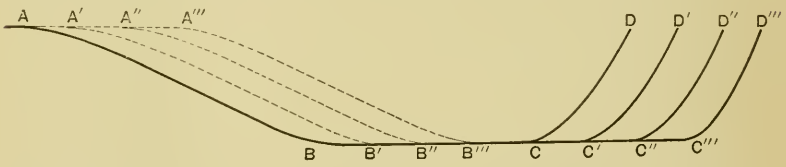

FIG. 22.

very great and important levee, or for the saving of valuable property, as on the front of a city. As a part of a regular plan for continuous river improvement, its expense and the necessity for repairs seem to be a very serious drawback.

The erosion which takes place at the foot of the revetment is a well-known objection to the holding of banks in concave bends. In the improvement of the Rhone, previous to 1878, the concave shores were provided either with bank protection or with parallel dikes. The strong high-water current scoured the bed so deeply and produced so great a pool that during low-water stages the velocity of the current became insignificant, and the fall was concentrated at the shoals existing at the reversion points, a condition highly unfavorable to low-water navigation. The reason of this excessive scour seems not to have been satisfactorily explained. Surely the portion of the water in contact with the bottom will not scour any the more because the portion in contact with the revetment is prevented from scouring. The *rationale* of the process appears to be this: Before revetment and while bank erosion was progressing, the bottom scoured out at the same rate that it did after revetment, but the bank, being undermined,

fell in and made good the loss of material on the bottom. The net result of this process was a lateral erosion, whereby the bed of the river was shifted from right to left or the reverse. The point of attack of the current against the bottom was thus continually changed. The river which lately occupied the position *A B C D*, Fig. 22, has taken in succession the positions *A′ B′ C′ D′*, *A″ B″ C″ D″*, etc. Now, if the bank *C D* be held by revetment so that it cannot fall in, the current makes its attack always at *C*, and the space scoured out is not filled up, but is augmented by each successive invasion of the current. Mr. Starling.

The author has alluded to the importance of selecting a point for beginning the work. In the earlier applications of bank protection, care was taken to rest the upper end of the work under a bar, or at a point where the bank was stationary, so that the revetment should not be flanked or taken in reverse by erosion above its commencement. It was found that the precaution was sometimes carried to excess. By the time 6 000 lin. ft. of work had been completed, perhaps half of it was rendered unnecessary by the advance down stream of the bar at its head. At Greenville, in 1891, when the town was threatened with destruction by a rapidly caving bank, it was decided to depart from the ordinary method and begin at the lower end of the caving reach, where the destruction was greatest and the necessity for action immediate. This course was accordingly adopted, and 6 000 lin. ft. of mattress was laid from below upward. It was feared that there would be a very serious loss of material at the upper end, but the apprehensions were not justified by the event, the damage being small. The bank caved away for a short distance above the head of the revetment and carried away 50 or 100 ft. of the latter. The pocket thus formed produced dead water, and the caving then ceased. Next year 4 500 ft. more were laid in the same way and with the same result. The success of these experiments induced the Mississippi River Commission to proceed in the same manner at Lake Providence in 1895, and doubtless this will be a well-recognized feature of practice in the future.

D. M. Currie, M. Am. Soc. C. E.—The author mentions having tried for years the form of protection used near St. Louis, termed the woven mattress, but finally notes its failure on the lower river, with the rest, without mentioning its success near St. Louis. This omission might be misleading to members not familiar with the facts. The woven mattress has been the standard form of construction for the protection of the subaqueous slopes of banks between St. Louis and the mouth of the Ohio River continuously since its adoption in 1881. The length of bank protected by this form within this period aggregates about 23 miles, and no protection has been lost or damaged on account of the failure of the mattress, showing that the form is efficient under the existing conditions, and emphasizing the difficulties met in the protection of the banks below Cairo. Mr. Currie.

Mr. Price. W. G. Price, M. Am. Soc. C. E.—The first submerged spur dikes used on the Mississippi River were constructed in the Gouldsboro' Bend of New Orleans Harbor, in 1884, and were designed by Col. Amos Stickney. These dikes have now been in place eleven years, and have withstood successfully a very swift current, which sets in very strong against this bank, where rapid caving was taking place before they were constructed.

At other places where this system has been used, and where it has not been so successful, the original plan has not been followed, and a cheaper and weaker one has been substituted. It is possible that the partial failure is due to this change of plan.

The dikes at Gouldsboro' were so constructed that all permanent fastenings were of wood, so that all the metal fastenings which were used in sinking the mattress and cribs might be removed, and the structure would still be strongly bound together. These dikes also had a mattress apron, which extended 119 ft. down stream from the top crib, to prevent the eddy which forms below the crib from cutting away the bank. The dikes built in the Third District and at Carrollton, in New Orleans Harbor, had the length of the mattress apron very much reduced in width. The two dikes built at Plaquemine in 1889 had a mattress apron 70 ft. wide, and after these dikes had been in place nearly two years, a careful examination of them was made by the writer, who found that the bank was being washed out below and close to the down-stream edge of the mattress, which was a sure indication that the apron was not wide enough to protect the bank from the eddy caused by the crib. The next three dikes, which were built at Plaquemine in 1891, had the apron reduced to a width of 65 ft., and the mattress and crib did not extend far enough out into the river to have a footing on the level bottom from which to brace up the bank. This change in the plan of the dikes was made, not with the expectation of making them better, but to reduce their cost. It is the opinion of the writer that this change of plan was the cause of the serious caving at Plaquemine which followed the construction of these three dikes, and was the cause of the failure of the dikes at Carrollton and in the Third District of New Orleans Harbor to protect properly the bank between them. The widths of apron required for various heights of crib and for different radii of curvature of river bank have not been precisely determined, but evidently it is not safe at any location to make the apron much narrower than those at Gouldsboro'.

The writer believes that the practice of using fastenings of metal in dikes and mattresses should be discontinued. It is necessary to use some metal fastenings to hold the cribs and mattress in place while sinking them, but they should not be essentially permanent parts of the structure.

Some years ago the writer had a chance to note the effect on plain

and galvanized wire of corrosion and the impact of particles of sand in suspension in a swift current in the upper Mississippi River. Where the current was not swift enough to carry sand against it, the plain steel annealed wire, No. 12, was corroded in about seven months so as to reduce the section one-half. In a swifter current, which carried sand in suspension, a No. 10 galvanized iron wire was reduced in section one-half in about three months, and the zinc was cut off in about 10 days. The wire was lifted out of the water so it could be seen once every day, and while the current was strong enough to carry sand in suspension no rust was found on it; which indicated that the entire reduction of section was caused by the impact of particles of sand. Mr. Price.

The mattress used in the dikes at Carrollton was bound with frames of sawed pine timber. Cottonwood timber would be just as good for this purpose if it were used while green, and it would be much cheaper. The frames extended in a direction normal to the bank line, so that they held the rock from being rolled down stream by the current. Willow poles, to hold the rock from rolling down the steep bank, were wired to the frames. After the wire has been destroyed by corrosion, the poles will give way. A better form of construction would be to use sawed timber in place of the poles, which could be fastened with wood pins to the upright pieces which project above the frames. This form of mattress is flexible, and will bend with its weight of rock so as to fit the uneven bed of the river; and if a part is broken down by excessive bending, it will not go to pieces. Great flexibility in a mattress is not necessary. The immense amount of wood, leaves, grass, roots, etc., which moves along the bottom of the Mississippi River will rapidly clog up any opening there may be under the up-stream edge of a thick mattress, and all other cavities down stream will then be quickly filled with sand.

The submerged spur dikes at Greenville were constructed of willows and poles, and bound together with wire and spikes, and they are liable to go to pieces after the metal fastenings have been destroyed.

The woven mattress is weak only along its edges, where the willows are bound with wire. The first break probably occurs at the deep-water edge, where the wire fastening is destroyed, and the swift currents bend the loosened willows down stream so as to release the first weaving pole, which is soon carried away, and then the next weaving pole is released, and the action continues until the mattress is gradually destroyed. That faults have occurred in the deep-water edge of the woven mattress probably does not indicate a lack of flexibility, but does indicate that the wire fastenings have been destroyed. After willow wood has been submerged for a few months it becomes heavier than water, so that when a mattress is being carried away no material comes to the surface. The mattress used in building the submerged

Mr. Price. spur dikes at Gouldsboro' was woven, but the edges were strengthened with a continuous crib filled with rock.

The fascine mattress as now used in the Mississippi is bound together with galvanized iron and silicon bronze wire. In swift currents it is possible that the sand will cut all this wire, so as to release the binding poles, and thus cause its destruction.

The mattress and cribs used in building the two dams across the Atchafalaya River, at Simsport, La., were all bound together with pins of wood, and for six years they have withstood a current over them which powerful steamships cannot stem at high water, except by passing up close to the shore. If the material in these dams had been bound with wire of any kind, they probably would have been destroyed within one or two years after they were completed.

Mr. Noble. ALFRED NOBLE, M. Am. Soc. C. E.—The paper is an extremely interesting history of the methods used for the protection of the banks of the Mississippi River below Cairo. In their development there has been little change in the general features, but the work has been steadily increased in strength and cost, the frail constructions first adopted having proved unequal to the task imposed. In the nature of things a large amount of experimental work had to be done; the engineers had almost no precedents to guide them in dealing with the prodigious problem of the regulation of the Mississippi, and it is not surprising or discreditable that so much of the work failed. If there had been no failures the conclusion would have been irresistible that much money had been wasted by constructions unnecessarily strong and expensive.

The tables of cost are of much value, but this value would be greatly increased if a proper allowance for plant were included so that the figures would represent the actual outlay. On this class of work a large floating equipment is required; in the moist climate of the lower Mississippi decay of this equipment goes on rapidly every month of the year, and the life of a barge is short. It will require extensive repairs in four or five years amounting almost to rebuilding, and a large annual expenditure afterward until abandoned. A proper allowance for the cost and maintenance of plant would probably increase the unit prices given in the paper at least 20 per cent.

It is to be regretted that the paper does not give a more complete statement of the results of bank protection at a few of the points where the most persistent work has been done. For instance, at Hopefield Bend just above the city of Memphis, work has been in progress nearly every year since 1883. All of the work done previous to 1890, it appears from the paper, was replaced before 1894, except in the upper end of the reach, where a sand bar had formed in front of the work and protection was no longer needed. A statement of the total cost at this locality, the amount of bank recession after the first protection

work was placed, and the cost of the work destroyed by successive floods, would give a vivid idea of the difficulty of the problem and some notion of the immense expenditure that will be required for the complete regulation of the stream, and might suggest doubt as whether the end to be gained would justify the outlay. However this may be as to the general question, there is no doubt as to the necessity of bank protection at certain points where large local and important public interests are so involved that the execution of improvements by local authorities is impracticable. An instance is afforded by Vicksburg, where a shipping and distributing point of considerable importance was nearly shut off from the river by a cut-off and the further travel of the channel was checked by the protection works at Delta Point. A still better illustration is given at Memphis, a place of much greater importance. Mr. Noble.

The city of Memphis is built on a bluff of a height of about 100 ft. above low water in the river. It is the only point for a long distance where the river flows against the bluff. At the upper end of the city is the mouth of Wolf River on which are located several saw mills, supplied with logs in rafts from the Mississippi; near the mouth is the transfer landing of the Little Rock and Memphis Railroad; below this the bank of the Mississippi is lined with warehouses for a distance of about half a mile, followed by a paved levee for the landing of steamboats, extending for a distance of another half mile, the two sections covering the business portion of the city. The Memphis Bridge is 1⅜ miles below the levee; about midway in this section are the terminal grounds of the Kansas City, Fort Scott and Memphis Railroad and its allied lines, situated on the bluff and having a river front of about half a mile. Immediately above the city is the stretch of river known as Hopefield Bend; in this stretch the course of the river is nearly normal to the general course of the valley and to the direction of the city front, and hence the current strikes squarely against it. Prior to 1878 the current struck the city front at its upper end, making it necessary to build the protection works described in the paper; at that time, however, the river was eroding the right bank in Hopefield Bend, or, in other words, this stretch of river was moving laterally down the valley. If not checked the result would be the destruction of Hopefield Point and the city would be shut off from the river. The revetment of the caving bank was undertaken to prevent this and to preserve the harbor.

In order to relieve the force of the current against the upper end of the city front it was deemed advisable to commence the revetment at the upper end of the stretch and carry it down stream from year to year. This would permit the cutting away of the bank in advance of the revetment; and the current would strike the Memphis front farther down stream and at a rather better angle. It was intended to build the revetment at the proper rate to stop the cutting and hold the bank

Mr. Noble. when the pressure against the upper end of the city front was somewhat relieved, but before any injurious filling could result in front of the transfer landing, the warehouses or the levee.

The proper execution of this plan required control of funds for completing the work at the exact time when the erosion of the bank in Hopefield Bend had reached the proper limit. At the critical period no funds were available, the cutting progressed rapidly with a corresponding fill on the opposite bank, and a sand bar, bare at a medium stage, formed in front of the city from the mouth of Wolf River to the paved levee and over-lapped its up-stream end. The force of the current was thrown against the bluff below the protected front, causing the loss of valuable ground, and in consequence a fund was raised by citizens of Memphis and expended under the supervision of the officers of the Mississippi River Commission for extending the protection down stream. The property of the railroad company referred to was below the new work and a considerable area fell into the river.

At the site where the Memphis Bridge was afterward built, there was formerly a strip of low ground several hundred feet in width under the foot of the bluff. With the conditions up stream changed, this began to wear away. During the winter of 1889–90 it had disappeared entirely as far down stream as the bridge line and a considerable quantity of earth had fallen from the bluff within 300 ft. of the bridge.

In order to stop this cutting in the immediate neighborhood of the bridge the chief engineer, George S. Morison, Past-President Am. Soc. C. E., decided to adopt, with some modifications, the method then in use by the Mississippi River Commission for the protection of cutting banks. The failures of river protection works which had already occurred pointed to the necessity of making the work stronger and loading it more heavily with rock, the latter remedying, to some extent, the want of flexibility in the mattresses and forcing them more closely to the bed of the stream. The mattresses were also made much heavier, the quantity of brush being nearly doubled and both longitudinal and transverse wire cables were used more freely.

The upper end of the protection work is 1 080 ft. above the bridge, the lower end 240 ft. below the bridge, making a total length of 1 320 ft.; the protected area is about 365 ft. wide, extending from high-water mark to deepest water in the channel, or very nearly to it. The conditions are in some respects more favorable than in Hopefield Bend, the ground not being alluvial silt, but the under portion of a bluff generally of friable clay with much gravel, which diminishes the danger, both from under-cutting and from scouring through the mattress. There is, however, a stratum of sand a short distance above low water, which yields readily to the current wherever exposed and explains some peculiar features in the caving of the bluff.

The work was undertaken in the fall of 1890. The equipment was

similar to that used on government work; the mooring boats and mattresses were anchored mainly with boxes filled with rock, shore anchors being used only for the shore edge. The anchor lines were 1¼-in. wire cables. It had been intended to weave the subaqueous mattress in one piece 250 ft. wide and 1 300 ft. long, but a succession of rises made it necessary to sink it in four sections, the accumulation of drift against the mooring barges straining the anchor lines so severely in each case that there was imminent danger of losing the portion woven. The several sections were made to overlap each other, and the loading along the lines of junction was made specially heavy. Although the mattresses were woven in the manner which has proved so unsatisfactory at Hopefield Bend, the work has been entirely successful, a result which is believed to be due, not only to the more favorable ground at the site, but to the more substantial construction of the mattresses. Mr. Noble.

In the interval of two years between the date of the survey of the bridge site and the commencement of the construction of the bridge, the west bank receded about 200 ft., and the west pier, originally located at the bank, now came in the river. It was desired, not merely to prevent farther cutting, but to restore the former shore line. The method adopted to secure this, which proved entirely successful, was to build along the line to be restored, a screen mattress hung up by one edge to a trestle, its up-stream end 1 080 ft. above the bridge, its down-stream end 75 ft. below it, making a total length of 1 155 ft. The up-stream end was curved toward the existing bank and the small opening closed by a dike of brush and stone. The top of the dike and of the screen was above ordinary high water. The screen mattress was loosely woven and was designed to permit the passage of water through it with a reduced velocity, inducing the deposit of the greater part of the material held in suspension.

In carrying out this plan a foundation mattress was first woven and sunk on the bed of the river, the whole length of the proposed screen. It was 150 ft. wide and extended outside of the screen line about 100 ft. This was heavily loaded with rock, particularly on the outer edge, so that if under-cut it would follow the cutting closely. The upper portion of this mattress was made 250 ft. wide and extended 350 ft. above the cross dike with its in-shore edge as close to the bank as it could be floated.

The next step was to build a light pile trestle along the proposed screen line, driving the piles through the foundation mattress which protected them effectually from scour. The screen mattress was then woven, one edge hung to the top of the trestle on the river face, the width of the mattress being 25 ft. to 40 ft. greater than the distance from the top of the trestle to the ground so that a broad flap rested on the bottom; this was also loaded with rock. In order to farther check the

Mr. Noble. current near the lower end of the protected area, a short spur dike was built from the shore nearly out to the screen about 100 ft. above the bridge line; the top was left at first considerably below high-water level, but was afterward raised as the filling progressed. The effect of this work was apparent immediately after the first rise, and in two years the area behind the screen was filled nearly to its original level.

The screen mattress and trestle are temporary, of course, and the portion of the former above low water is no longer capable of resisting a vigorous attack of the river. During the last two years, however, the river has shown no tendency to encroach in that direction and while this condition continues no additional protection will be required, but if the bank should be attacked, the action is not likely to be violent, and it can be protected by the usual revetment.

An application of mattress protection, believed to be novel, was made in connection with the foundations of the two river piers, which were of masonry on pneumatic caissons. At the site of one of these piers the bed of the river was about 30 ft. below extreme low water; at the other, 10 ft. higher. The bed, which is of sand, to a depth of 50 ft., would scour if a caisson were lowered near it; the scour would be greatest at the up-stream end and would make it doubtful if the caisson could be landed safely on the river bottom. To guard against this contingency the chief engineer decided to protect the river-bed by a mattress at each pier site before bringing the caisson into position; the caisson to be landed upon it and sunk through it. The mattresses for this purpose were placed at both pier sites in 1889.

The pier site in deepest water was first taken in hand. The area to be covered was 400 ft. square. The weaving plant provided for a width of 240 ft. only, so that it was necessary to weave two mattresses to make up the required width.

The distance from shore, nearly 800 ft., as well as the fact that the main steamboat channel was in this opening, precluded the idea of anchoring the mattress and plant, either wholly or in part, from the shore: Boxes filled with stone were therefore placed in the river as anchors, to each of which two manilla lines 2 ins. in diameter were attached, one leading to the mooring barge, the other under the barge to the head of the mattress. The mattresses were woven in the manner fully described by the author. For the first mattress the ten anchors were 5-ft. cubes without projections. It was believed that they would sink into the bed of the river quickly and would be practically immovable. This did not prove to be the case. In the later course of the work box anchors were repeatedly moved from place to place, and there was no apparent difficulty in lifting them other than that due to their weight.

When the weaving of the first mattress was completed it was loaded evenly with rock until it barely floated; the up-stream end was con-

FIG. 1.

FIG. 2.

nected to the anchors, as already stated, by lines passing under the mooring barges. During construction it was held up to the surface of the water by lines leading to timber heads along the down-stream side of the mooring barges; after the mattress was loaded these lines were slacked off enough to permit hauling across the head of the mattress two barges placed end to end and loaded with rock. When in position they were parallel to the mooring barges and immediately below them, and connected with them by lines; in moving across the head of the mattress enough rock was thrown from the barges to sink it as far as the lines would permit, and sufficiently to allow the floating of the barges over it. When finally in position, with a large force of men on board to throw off rock, the lines connecting the rock barges and mooring barges were slacked off steadily and uniformly, the men casting rock on the mattress as rapidly as possible while the current carried the rock barges over it. When about one-third of the mattress had been passed over, the lines joining the mattress-head to the mooring barges, then severely strained, were cast loose simultaneously, and the head began to sink to the bed of the river, swinging on the anchor lines, the down-stream end of the mattress still floating, being not fully loaded. As the head sank the mattress offered increasing obstruction to the current and before it reached the river bed in 40 ft. of water, the anchors began to drag and mattress and fleet moved down the river. When the head of the mattress reached the river-bed it increased the effective anchorage and the movement stopped; the current quickly forced the down-stream end under water, even before the rock barges had reached it. The whole operation of sinking had occupied less than six minutes. The movement is shown very well in Plate VI, Figs. 1 and 2, which are taken from the same point, the former before the movement commenced, the latter when it had stopped. The entire movement was 130 ft., and the head of the mattress was only 80 ft. above the center of the pier site instead of upward of 200 ft. as intended. The question arose at once whether it would be necessary to sink an additional mattress over the portion of the area intended to be protected, but still left uncovered. Inasmuch as the pier site would be actually protected, and as the stage of the river was low, with prospects of a continuance of the low stage, and, furthermore, as the season was growing late, while much work of this kind remained to be done, the question was decided in the negative. The building of the second mattress was then commenced according to the original plan. The anchors were increased in size to 6-ft. cubes; frames were made for the top and bottom of any convenient timber, usually 6 x 8-in. oak, to which the plank lining was spiked, the ends of the timbers projecting 10 to 20 ins. to give a better hold on the ground, as shown in Plate VII, Fig. 1. When the next mattress was sunk the anchors pulled slightly, and one or two of the lines broke, showing that the resistance

Mr. Noble.

Mr. Noble. of the anchors was very nearly equal to the strength of the lines. The mattress was placed at almost exactly the spot desired; the caisson for the pier was then placed in position and landed safely upon the mattress without special incident.

On account of the lateness of the season. it was decided to place a single mattress, 240 x 400 ft., at the site of the remaining pier. This was done without difficulty, but after anchoring the caisson at the site a sharp rise of the river occurred, reaching a 23-ft. stage, and the velocity of the current became nearly 5 miles per hour. While the floating caisson was being built up and filled with concrete, the water was driven under its up-stream end with great force, and a scour of 4 or 5 ft. occurred through the mattress, showing that for those conditions the protection was imperfect. The mattress answered its purpose, however, and after landing there was no difficulty in leveling the caisson and sinking it.

While this caisson was afloat the strength of the anchorages was severely tested. The anchor lines were steel wire cables 1¼ ins. in diameter, and were attached generally to boxes of 7 ft. cube, similar in construction to those described. In adjusting the strains on the anchor lines they were frequently pulled with a 2-in. diameter manilla line rove in triple blocks, with the fall carried to a winch head on an ordinary Mundy hoisting engine. In order to get the strain desired, it was customary to take a run with the engine, then tighten the line on the winch and stall the engine. None of these anchors yielded under this severe pull. The anchors were sunk in about 30 ft. of water, and the anchor lines were 800 to 1 000 ft. long.

The piers of this bridge are founded in a hard, compact clay, underlying about 50 ft. of sand, and would doubtless carry safely the load imposed if the sand were scoured away; but obviously if the scour could be prevented and the frictional resistance of the sand made available for resisting settlement, the factor of safety would be increased. Furthermore, while the danger of scouring under the foundations in this material is exceedingly small, still an appreciable degree of security would be added if scour were prevented; the mattresses just described would accomplish this so long as they were preserved intact, but some doubt was felt in regard to this, because they had been subjected to very severe usage. In the main channel the velocity of the current had reached 8 miles per hour, and soundings showed that the edge of the mattress, if still in place, had been sunk about 20 ft. by under-cutting. For this reason it was thought best to sink a new mattress at each pier, and to load it very heavily with large stone; these mattresses were built in the same manner and with the same plant as before, except that the two weaving boats, instead of being placed together, end to end, were separated by an interval of 20 ft., which was closed by a temporary staging. When the mattress was woven down-

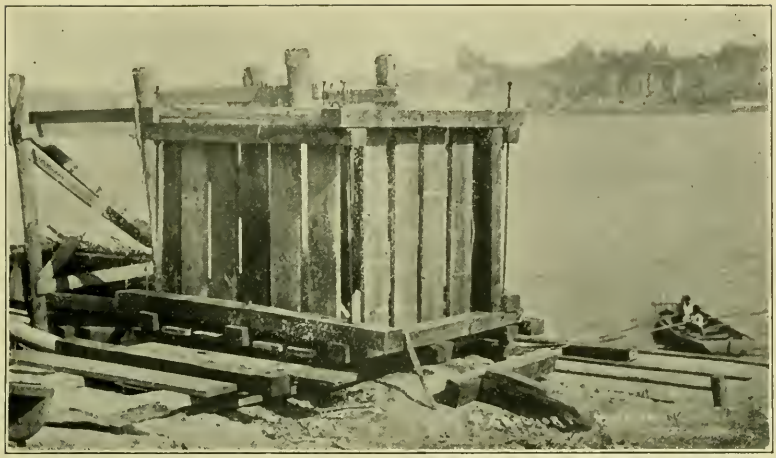

FIG. 1.

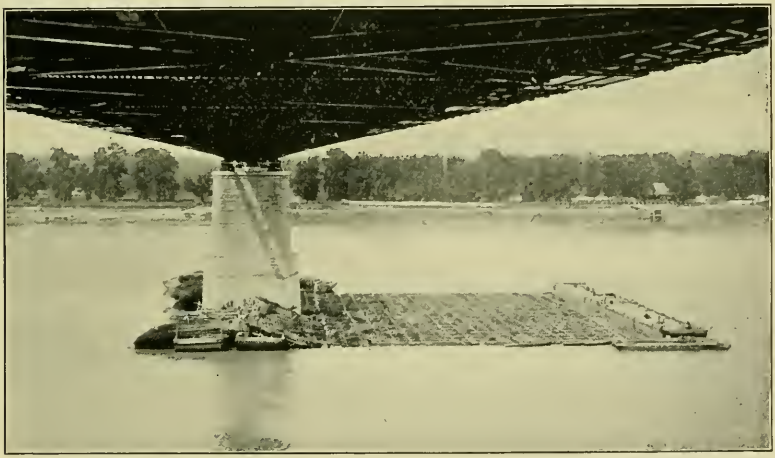

FIG. 2.

stream to the pier, the staging was removed until the pier had been passed. In this manner a continuous mattress was made at each pier 260 ft. wide by 400 ft. long, with a hole in the center just large enough to enclose the pier. The view shown in Plate VII, Fig. 2, was taken while the staging was removed and the weaving barges were passing the pier. After being sunk, the mattresses were covered with heavy quarry rock, running up to 2 or 3 tons in weight, to a depth of upwards of 5 ft. at the piers, and tapering off to 1 ft. at the edges of the mattresses. Soundings along the bridge line, which are still made regularly, indicate that this work is at yet uninjured. Mr. Noble.

The average cost of the mattress work at the Memphis Bridge per square of 100 sq. ft., not including the heavy riprap at the piers, was as follows:

1.1 Cords of brush and poles	$1 56
11.1 Lbs. wire and wire cables	46
1.1 Tons riprap	1 53
Sundry materials	09
Plant	1 90
Labor	2 43
Cost per square	$7 97

The large cost of plant is noticeable. A part of the floating equipment was not required for other work, and was wholly chargeable to this account, while a considerable portion was charged in part to other accounts. On the other hand, the plant was in use only two or three years; some of it was built, the remainder was bought at market rates, and at the end of the work all of it was sold at low prices as quickly as possible to avoid maintenance charges, the conditions being practically those of a forced sale. The cost of plant was, therefore, probably greater than on bank revetment work done by the United States Government. The figures cover every item of expenditure, except engineering expenses, which would not exceed 2 or 3 per cent.

H. St. L. Coppée, M. Am. Soc. C. E.—The difference in conditions existing in this country and Europe are quite marked. In the latter, as Mr. Starling states, the brush has to be cultivated with considerable trouble and care, while here in spite of the constant denudation for the purpose of revetment, the bars still contain sufficient quantities to supply the demand, a continual natural growth replacing that destroyed. Mr. Starling's explanation of the manner in which the bank caves where unprotected is undoubtedly correct, but his statement that the scour at the bottom of the slope beyond the mattress will not be augmented, is considered incorrect by the author for the following reasons: Mr. Coppée.

Mr. Coppée. When the bank is unprotected and irregular, the force and velocity of the current is decreased, not only by the loss of energy expended in reducing the bank, but also by its endeavor to carry in suspension a large amount of the abraided material. If the bank in the bend is protected with a smooth continuous revetment, there will be less obstruction to the general flow, and consequently greater velocity; and the transverse and downward currents near the shore will act with greater freedom and force. In other words the currents will be stronger, and as the scouring force increases with the square of the velocity, the abrasion will be greater at the foot of the slope where the bank is susceptible.

In regard to the point of beginning for extensive reaches of protection work, though in Greenville Harbor it was selected at the lower end where the caving was most rapid, and carried up stream, no rule has been made other than the general principle now adopted of commencing the work at the point where the caving is greatest and working up or down stream as would seem most advisable under the existing conditions, depending on the existence in the reach of other revetment, the value of property to be saved, etc.

A brush pile dike somewhat similar to that mentioned by Mr. Tratman as in use on the James River was constructed at Vicksburg in 1887, for the purpose of intercepting a current carrying silt into the harbor. The materials of construction were timber piles, brush, poles, wire spikes and rock. A mattress dike was built in the Plum Point Reach, near Gold Dust Landing, Tenn.,* not unlike those used on the James, where pile driving was impracticable.

Suspended screen mats and curtains have been used in this country on the Missouri and Mississippi Rivers. On the lower reaches of the latter river they were not generally successful in causing accretion.

The author did not intend to create the impression that woven mats were not considered of value at many points, because they were abandoned in the more exposed reaches for the fascine type of subaqueous revetment. On the contrary, there are many localities and many rivers in which their use is the best of practice.

Mr. Price is mistaken in supposing the cause of failure of spur dikes or submerged spurs, where such occurred in the upper districts, to have been due to defective design or weak construction. In every instance of which the author has record, the fault has been in the weak soil, strong currents, great oscillation of the water surface changing the point of attack, etc. A spur constructed under the direction of the author at Greenville, Miss., in 1889, at the head of a system about a mile in length, was flanked and undermined by the current, but not destroyed, remaining intact, the river cutting a new channel behind it. In the Third District and Carrollton bends, New Orleans

* See the "Report of the Chief of Engineers," U. S. Army, 1894, p. 2890.

Harbor, caving took place in the intervals between the dikes. This may have been due to insufficient width of apron and length of dike, but it caused the following expression of opinion by the officer in charge: Mr. Coppée.

"Spur dikes without intermediate revetment have been successful in some straight reaches and on concave banks of large radius, but in the abrupt bends the dikes alone are only locally effective."

In mattress construction Mr. Price lays great stress on the supposed weakness of the wire fastenings, and does not account for the effect of the large mass of stone covering the submerged structure. Where the mattresses are placed, as a rule, there is but a small amount of fine sand and no coarse sand in suspension in the water to cause abrasion. The deterioration from oxidation is probably not as great on the submerged as on the upper-bank work. The author has examined old revetment in which the wire was intact that had been subjected to a constant and strong current for over ten years.

Though it is the general opinion that the fascine mat is more flexible than the woven or frame mat, it is not so much to its flexibility, but rather to its imperviousness and its greater strength that is attributed its vast superiority over other forms. The framed pin and pole structure used under the spurs at New Orleans, and in the sill at Red River, would hardly stand the strains in sinking to which the fascine form is subjected.

Mr. Noble describes the use of large mattresses in a manner not generally familiar to river engineers; that is, as the foundation at first and afterward the protection of the caisson on which a bridge pier was constructed. In river work it is not often necessary to sink mattresses in the bed of the stream far distant from the bank, but when it is, the method adopted by Mr. Noble is generally followed, large wooden frame and rock anchors being used in place of shore fastenings.

The cost of plant, interest, deterioration, etc., was not given in the paper, because, though the value of the individual pieces of floating property could be readily ascertained, they are so differently distributed between different portions of the work in one season that it is practically impossible to tell what to charge to the different localities. The following list gives the pieces, with their approximate first cost, used by a bank protection party:

1 Steamboat, 200 tons, engine 17 x 6 ins., approximate dimensions, 130 x 25 x 5 ft.	$20 000
2 Mattress barges, 160 x 32 x 5 ft., each	4 500
2 Mooring barges, 160 x 30 x 5 ft., each	4 000
8 Brush barges, 120 x 30 x 6 ft., 250 cords capacity, each	3 000
8 Stone barges, 120 x 30 x 6 ft., 350 yards "	3 000
2 Line barges, 75 x 15 x 4 ft., each	1 000

Mr. Coppée.

2 Quarter boats and outfit, 120 x 23 x 5 ft., capacity 150 men, each	$4 000
1 Small office quarter boat	2 000
1 Hydraulic grader, including all hose and fixtures	20 000
6 Skiffs, each	20

The number of stone and brush barges necessary is governed by the distance to point of supply. The cost of lines and cables and the number required are too variable for estimation within reasonable limits.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.

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TRANSACTIONS.

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No. 776.

FLOW OF WATER IN 48-IN. PIPES.

By DESMOND FITZGERALD, M. Am. Soc. C. E.

PRESENTED FEBRUARY 5TH, 1896.

WITH DISCUSSION.

The experiments here described were made in 1894–95 on the Rosemary inverted siphon, a part of the Sudbury Aqueduct supplying the city of Boston with water. As the pipes had been in use sixteen years it was thought that it would be of value to determine how much their capacity had been diminished by the increased friction due to the incrustation of the interior surfaces. A series of experiments was planned in 1893, but it was not until September, 1894, that all the preparations were completed, the weirs and piezometers erected, and the experiments begun. Many delays and difficulties were encountered from the fact that much of the work had to be done inside the aqueduct, which was in regular service, so that the flow could be stopped only for short intervals.

Description of the Siphon.—The Sudbury River Aqueduct is 17.4 miles long, and for the greater part of its length it is 7 ft. 8 ins. high and 9 ft. wide. At a distance of 11.7 miles from its head at Framingham the

water is carried across the valley of Rosemary Brook through two 48-in. cast-iron mains, 1 800 ft. long, laid side by side on a straight line in plan, and descending gradually into the valley to a grade about 48 ft. below the bottom of the aqueduct, forming an inverted siphon (see profile, Fig. 1). The changes of gradient are made with vertical curves. The ends of the pipes are furnished with gate-chambers covered with substantial masonry buildings. The pipes were laid in 1877, and were first put into service in 1878; they were of the usual hub and spigot form, cast in lengths of 12 ft. The pipes were coated with Dr. Angus Smith's coal-tar preparation. The joints were well made. Two diameters were measured at each of 37 stations; the mean diameter thus obtained was 3.998 ft., taken as 4 ft. An attempt was made to figure the diameter by filling the pipe with water which was first passed over a small standard weir. It was found impossible to arrive at as correct a result by this method as by measurements of the pipe.

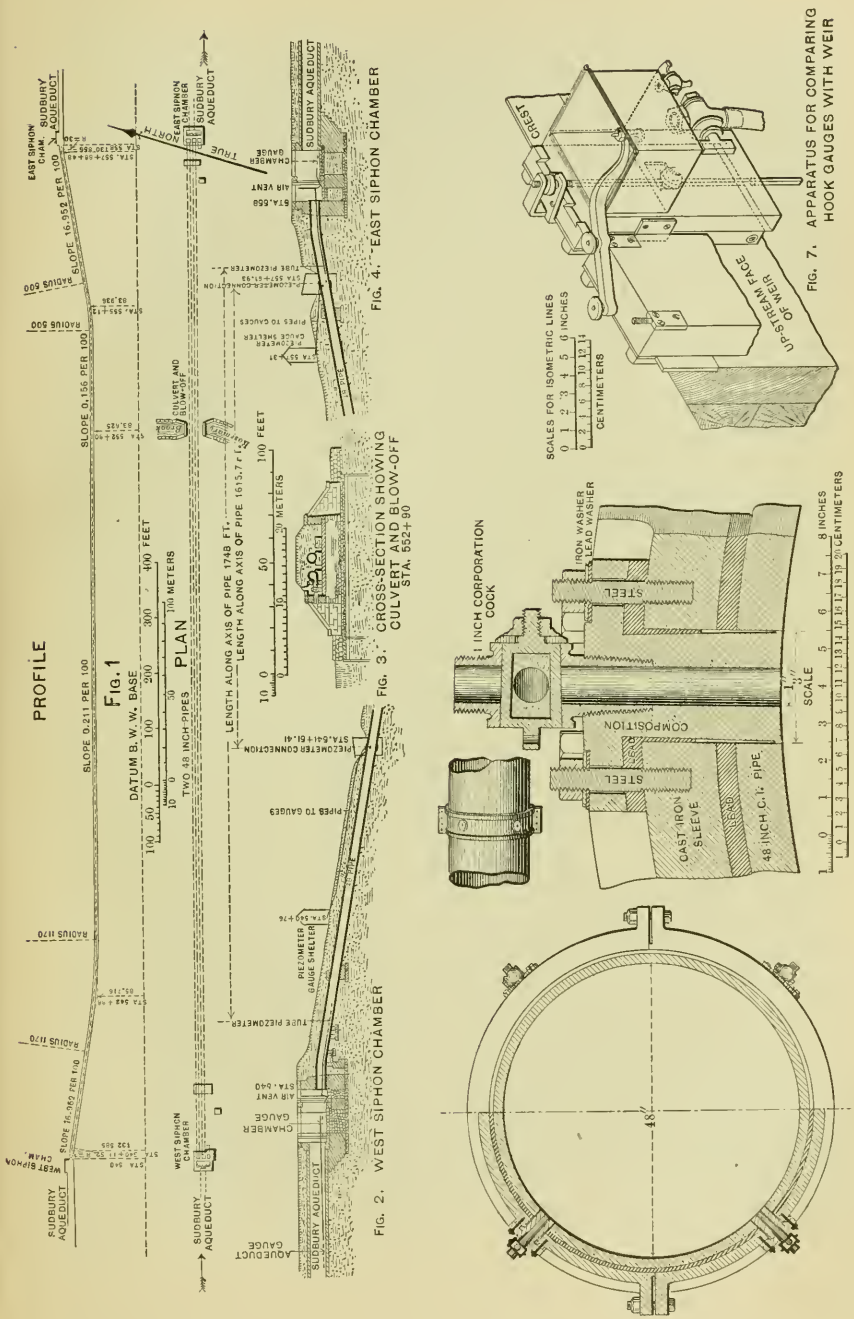
Before beginning the experiments, photographs were made by flash light of the interior of the pipes showing the tuberculated surfaces. Two of these photographs are reproduced on Plate VIII, Figs. 1 and 2. It was estimated that the tubercles covered nearly one-third of the interior surfaces, the bottom being more thickly incrusted with them, while the tops and sides of the pipes were cleaner.

Scheme of Experiments.—The losses of head were measured as near the extremities of the pipe as it was wise to place the piezometers, and in two ways, first, by a set of piezometers screwed into the pipes, and second, by tube gauges placed lengthwise on the bottom of the pipes and connecting with lead pipes extending out of the open ends to gauge chambers. The former method was more elaborately carried out and was considered the principal one. The latter had been adopted by F. P. Stearns, M. Am. Soc. C. E., in his experiments at the same place, and by using it in addition it was expected that a valuable check on the results would be secured.

It was found impossible to arrange a weir measurement in the aqueduct near the siphon which would admit of a greater velocity than about 3.7 ft. per second through one pipe. It was therefore determined to measure the flow up to this velocity through each pipe in turn; then to scrape all the tubercles from one pipe and experiment again upon the pipe as cleaned; then to erect two weirs at the terminus of the

FIG. 1.

FIG. 2.


PROFILE
Fig. 1
DATUM B. W. W. BASE
PLAN
TWO 48 INCH PIPES
SLOPE 0.211 PER 100
SLOPE 0.156 PER 100
SLOPE 16.552 PER 100
RADIUS 170
RADIUS 500
EAST SIPHON CHAM.
WEST SIPHON CHAM.
SUDBURY AQUEDUCT
CULVERT AND BLOW-OFF
NORTH
LENGTH ALONG AXIS OF PIPE 1748 FT.
LENGTH ALONG AXIS OF PIPE 1615.7 FT.
PIEZOMETER CONNECTION
PIPES TO GAUGES
TUBE PIEZOMETER
PIEZOMETER GAUGE SHELTER
AIR VENT
CHAMBER GAUGE
AQUEDUCT GAUGE
FIG. 2. WEST SIPHON CHAMBER
FIG. 3. CROSS-SECTION SHOWING CULVERT AND BLOW-OFF STA. 552+90
FIG. 4. EAST SIPHON CHAMBER
CREST
UP-STREAM FACE OF WEIR
SCALES FOR ISOMETRIC LINES
INCHES
CENTIMETERS
FIG. 7. APPARATUS FOR COMPARING HOOK GAUGES WITH WEIR
1 INCH CORPORATION COCK
IRON WASHER
LEAD WASHER
STEEL
COMPOSITION
CAST IRON SLEEVE
LEAD
48 INCH C. I. PIPE
SCALE
FIG. 6. SECTION THROUGH PIEZOMETER
48"
FIG. 5. HALF SECTION AND HALF-ELEVATION OF SLEEVE

aqueduct at Chestnut Hill Reservoir about 5.3 miles distant, and connect the weir measurement at the siphon with the weir measurement at the terminal chamber by means of an extended series of observations which would give a comparison between them at different rates of flow up to about 45 cu. ft. per second. The siphon weir was then to be taken out in order to secure as high a velocity as possible in one pipe, the discharge being measured by means of the weirs at the terminal chamber.

This general programme was carried out. The new weirs of course held back the water in the aqueduct, and it required a careful adjustment of the hydraulic gradient to arrange for the desired flow; but the coefficient of friction of the aqueduct was well known, as was the approximate coefficient of the pipes after the first series of experiments had been completed. Observations were begun on September 4th, 1894, upon the south pipe, tuberculated, and were continued on this pipe at various times up to and including October 6th. Velocities ranged from about 3.5 ft. per second to rather less than 0.5 ft. per second, and the experiments were generally repeated several times for each velocity. On October 18th the flow was changed to the north pipe, tuberculated, and this pipe was experimented upon with the same series of velocities as the south pipe.

The north pipe was cleaned November 12th, 13th, 14th and 15th by removing the tubercles completely from the interior. A photograph was then taken of the surface as cleaned (see Plate IX). It was found possible to remove tubercles without injuring materially the original coating underneath them, and, as will be seen by an examination of Plate IX, the original condition of the pipe was practically restored. The tubercles generally had central points or very small spots of attachment to the iron of the pipes. At these points the coating was lacking of course; but around them the tubercles spread over the surface of the coating, which remained in fair condition beneath. The original capacity of the pipe to pass water under a given head was nearly restored, the increase amounting to 30% at ordinary velocities. This was accomplished at small expense. There were 22 619 sq. ft. of surface scraped, swept and cleaned, with 57 days' labor, the space cleaned per man per day being 396 sq. ft. of surface, or about 32 lin. ft. of pipe. The labor was subdivided as follows: 32 days scraping tubercles; 21 days sweeping and washing the

PLATE IX.
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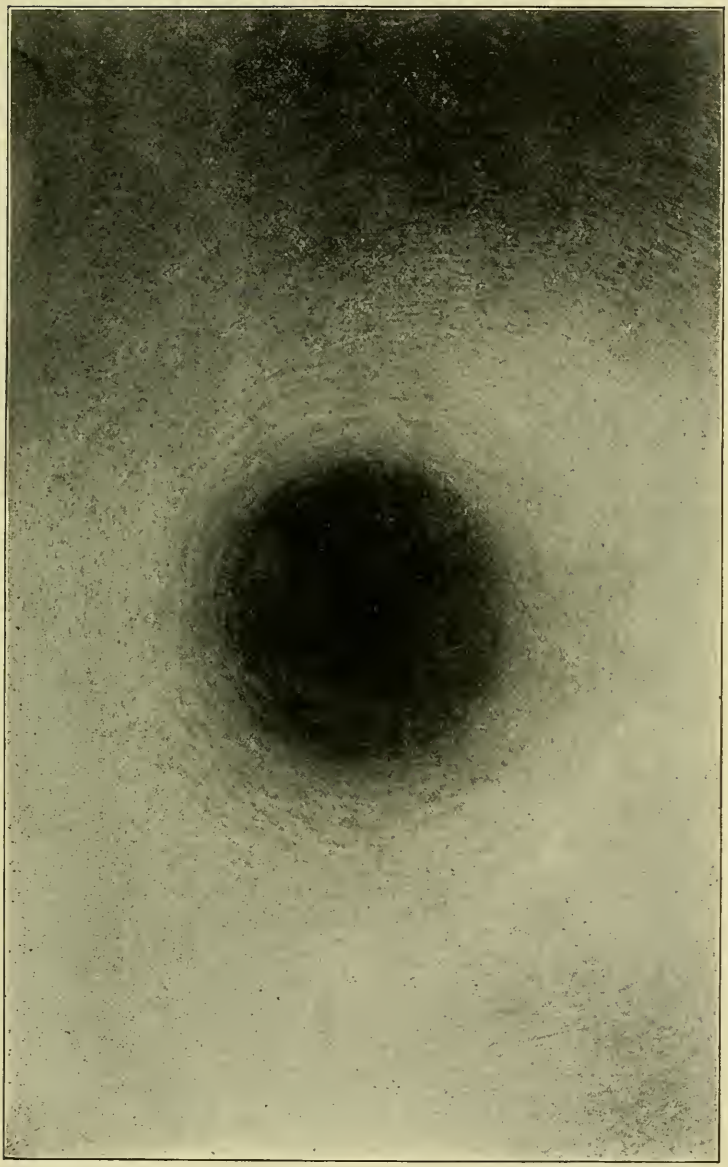


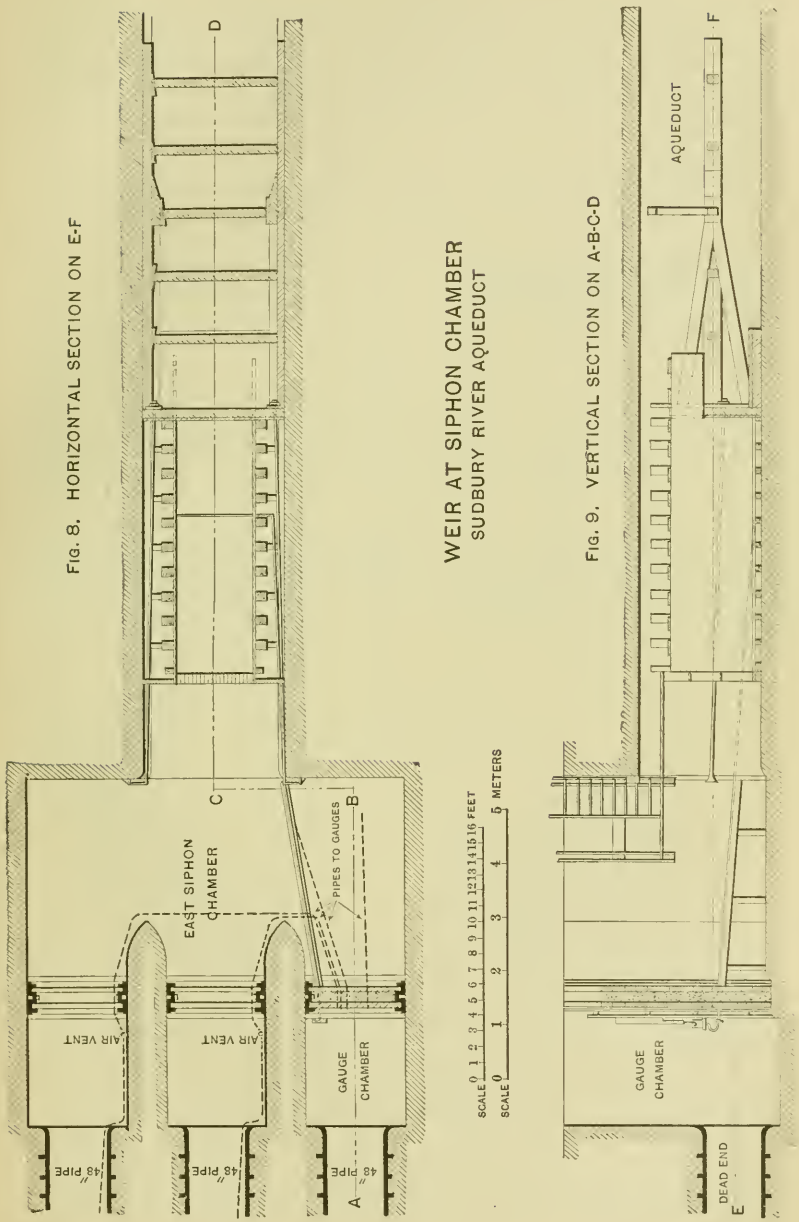


Fig. 8. HORIZONTAL SECTION ON E-F
WEIR AT SIPHON CHAMBER
SUDBURY RIVER AQUEDUCT
Fig. 9. VERTICAL SECTION ON A-B-C-D
D
C
B
A
E
F
EAST SIPHON CHAMBER
PIPES TO GAUGES
AIR VENT
AIR VENT
GAUGE CHAMBER
48" PIPE
48" PIPE
48" PIPE
AQUEDUCT
GAUGE CHAMBER
DEAD END
SCALE 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 FEET
SCALE 0 1 2 3 4 5 METERS

pipe, and 4 days wheeling material. The scraping for a width of 2 ft. at the bottom was done with old round-pointed iron shovels; for the rest of the pipe with wooden scrapers made of oak, great care being taken not to disturb the tar coating. About two cart-loads of tubercles were taken out, beside what was flushed out of the blow-offs.

Beginning on November 30th, the experiments were continued on the cleaned north pipe at velocities ranging from 3.6 to less than 0.5 ft. per second. Early in November the terminal chamber weirs were erected and compared with the siphon weir (see Table No. 5), after which the siphon weir was removed and experiments were resumed on the cleaned north pipe, and extended to velocities of 7.25 ft. per second, which was the largest amount of water that it was practicable to pass through one pipe. On January 23d, 1895, the experiments at high velocities on the south pipe, tuberculated, were completed, but it was found practicable only to carry these velocities to 5.5 ft. per second.

The piezometer observations are recorded in Tables Nos. 3 and 4, the former including those piezometers only which were screwed into the sides of the pipes, and the latter the tube piezometers lying on the bottom of the pipes.

Measuring Apparatus at the Siphon Chambers.—The siphon chambers, as may be seen in Figs. 2, 4, 8 and 9, are built so that a third 48-in. pipe can be added in the future. It was determined to use the compartments provided for this third pipe as gauge chambers. They were made perfectly tight by means of double sets of stop-planks with puddle filling. The tube gauges were led into these compartments, and in the case of the east siphon chamber a pipe from the weir was carried into the same compartment to a small gauging box to which a hook-gauge was adjusted. Great care was taken to have these pipes rise by uniform grades to their gauges, and, where a summit was unavoidable, to put in a vent pipe carried above the water-level. In this way serious errors, which are often made in the readings of piezometers and weir gauges, due to collections of air, were avoided. The lengths of pipe under observation were 1 748.1 ft. of the north pipe and 1 747.96 ft. of the south pipe, those being the distances, referred to the axes of the pipes, between the middle points of the brass tubes.

Piezometers.—The piezometers that were screwed into the pipe from the outside were arranged in the following manner: A point was selected at the west end of the siphon sufficiently remote from the entrance of the pipe to allow the water to attain regular motion. This was 151.41 ft. from the upper end of the pipe. The east piezometers were inserted at 38.07 ft. from the lower end. The distance between these points, measured along the axis of the pipe, was 1 615.7 ft. Four openings were made in each pipe at each of these points (sixteen

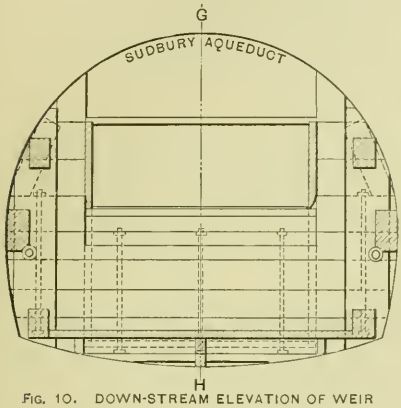


FIG. 10. DOWN-STREAM ELEVATION OF WEIR

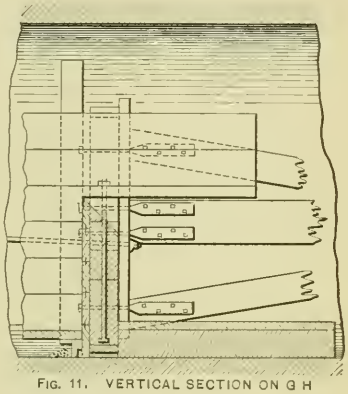


FIG. 11. VERTICAL SECTION ON G H

SCALE 0 1 2 3 4 5 6 FEET
SCALE 0 1 2 METERS

DETAILS OF
WEIR AT SIPHON CHAMBER

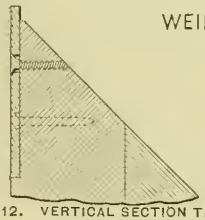


FIG. 12. VERTICAL SECTION THROUGH CREST

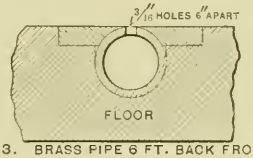


FIG. 13. BRASS PIPE 6 FT. BACK FROM CREST

SCALE 0 1 2 3 4 INCHES
SCALE 0 1 2 3 4 5 6 7 8 9 10 CENTIMETERS

holes in all), and as the welfare of the city depended upon the maintenance of the flow, sleeves were first put around the pipes where the holes were to be made. A tool was devised by which 3-in. holes were bored through the sleeve and pipe at the four points of section, as shown in Fig. 5.

Into these holes piezometers were inserted and secured, so that the axes of the tubes were at right angles to the direction of the flow,

These connections were accurately finished to the curvature of the pipe. They can be distinguished in Plate VIII, Fig. 2. The incrustation was removed from the interior of the pipe around the place of attachment, so that this portion of the surface was smooth. Into the outsides of these piezometer connections 1-in. corporation cocks were fitted to which lead pipes were soldered. These lead pipes from opposite sides of the 48-in. pipes were connected in pairs and then carried to the gauges, which were conveniently located for reading the extreme variations of head, and were provided with shelters for the observers. Care was taken to have no summits or depressions on these pipes.

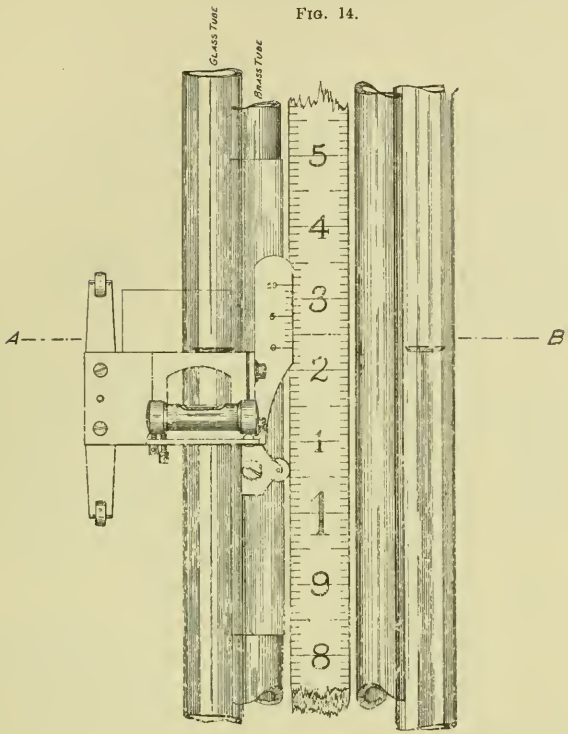
Tube Piezometers.—The tube piezometers which lay in the pipes were of $\frac{5}{8}$-in. brass tubes, pierced by 14 $\frac{3}{16}$-in. holes, spaced 6 ins. apart on centers. The tubes were closed with pointed oak plugs and were secured to the bottoms of the pipes and the spaces about the tubes filled with Portland cement.

Gauges.—Each shelter contained a gauge arranged as shown in Figs. 14 and 15. This gauge was set for reading the two glass tubes containing the water columns, which were about 1 in. in internal diameter. These glass tubes could be connected with the lead pipes from either of the 48-in. pipes as desired. Behind the glass tubes were brass pipes for the verniers to slide upon. The scales were specially graduated by Gurley, and the verniers were made by Buff & Berger. The readings were recorded to thousandths of a foot. The lead pipes were furnished with stop and waste cocks, both at the connections with the 48-in. pipes and at the bottoms of the gauges, so that the observers could frequently test the water columns, to make sure that they were free from obstructions.

Weir at the Siphon.—The general arrangement of the weir at the siphon with details is shown in Figs. 8, 9, 10, 11, 12 and 13. It was 5 ft. long, 3.04 ft. high and approached by a channel 16 ft. long. It was securely built into the aqueduct and was tied by iron rods to the gate-chamber, so that it could not move, and was braced with heavy timbers on the down-stream side. It was provided with a screen to smooth the approach of the water to the weir. The head was read at a point 6 ft. above the crest by means of a brass tube laid across the channel of approach, laid flush with the bottom and

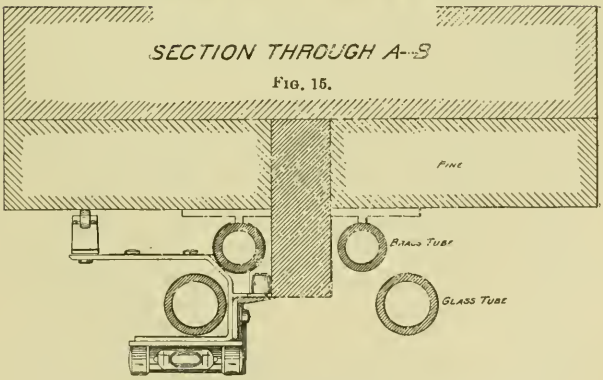
ELEVATION

Fig. 14.



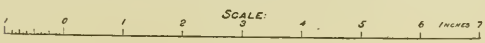
SECTION THROUGH A—B

Fig. 15.



PIEZOMETER GAUGE AND VERNIER

SCALE:



pierced with $\frac{3}{16}$-in. holes 6 ins. apart, smoothly finished, and connected with the measuring chamber as already described. A blow-off cock at the gauge, which was frequently opened, prevented accumulations of air.

Careful measurements of the weir were made before each experiment, and changes in form, due to swelling or distortion, were noted. This is necessary in the case of a wooden weir, for, even with the best of workmanship, changes are constantly occurring. Fig. 7 shows an improved apparatus for comparing the level of the weir with the hook gauge. The method is the same as that used by Messrs. Fteley and Stearns in their weir experiments, but the apparatus has been made more convenient by the special arrangements shown. The hook is first adjusted by a delicate spirit level to the level of the crest, the sliding tin box is then brought up under the hook, the cock opened and an approximate adjustment made of the water surface to the level of the hook, which is made complete by a slow-motion screw; the elevation can then be read at once at the hook gauge at the other end of the pipe without waiting for wave motions to subside.

Weirs at the Terminal Gate Chamber.—The Sudbury Aqueduct terminates at Chestnut Hill Reservoir with a large stone chamber provided with five compartments controlled by gates and stop-planks, and conducting the water into the reservoir or into other connections. The two west compartments connect with the reservoir. In these, two weirs were built, and the other connections were dammed off by clay dams made perfectly tight. The west weir had a length of 5.84 ft., and the east weir 6.32 ft. They were both at the same level and 3.65 ft. high. The position of these weirs with reference to the flow in the aqueduct was peculiar. But satisfactory results were obtained by extensive screening and the use of a liberal amount of brush compactly placed, as shown in Figs. 16 and 17. The sheets of water passing over these weirs, with the largest quantities flowing, were not perfectly smooth, but very nearly so.

Comparative Observations between the Siphon and Terminal Chamber Weirs.—The flow in the aqueduct was adjusted at 3 o'clock P. M. for the amount of water required, and at 10 o'clock the next morning, when the water was flowing uniformly throughout the length of the aque-

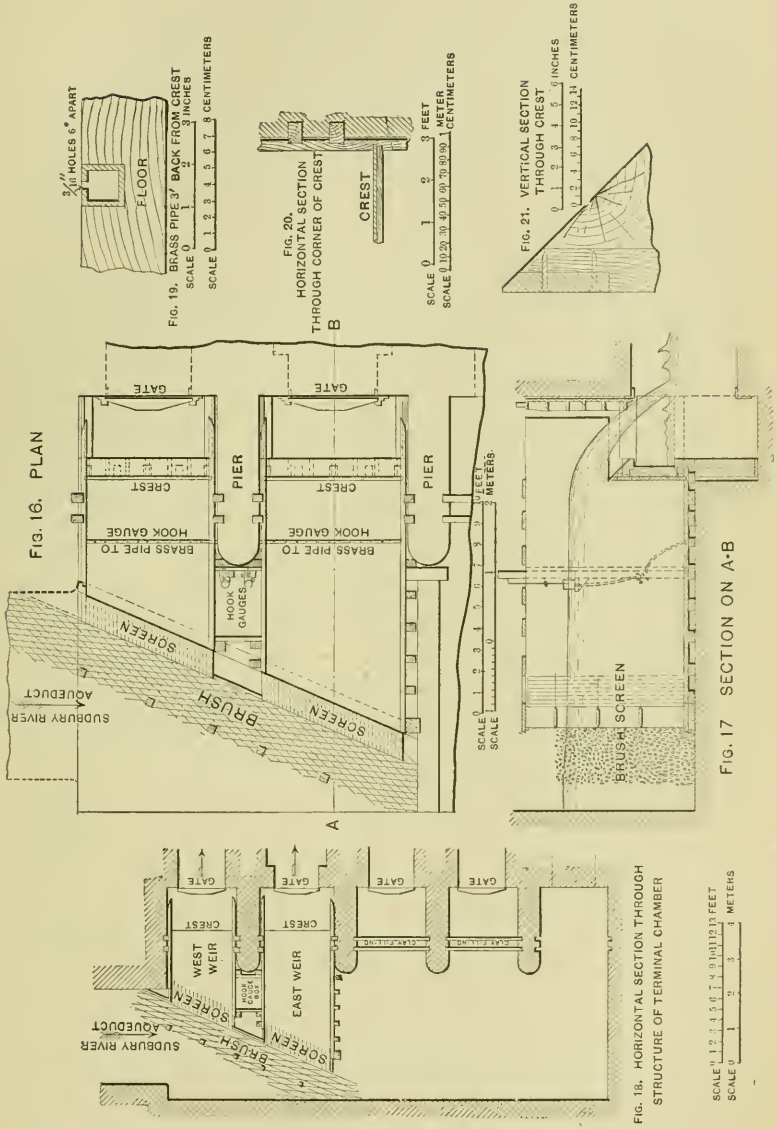


FIG. 19. BRASS PIPE 3" BACK FROM CREST
SCALE 0 1 2 3 4 5 6 7 8 CENTIMETERS
3/16" HOLES 6" APART
FLOOR
FIG. 20. HORIZONTAL SECTION THROUGH CORNER OF CREST
CREST
SCALE 0 1 2 3 FEET
SCALE 0 10 20 30 40 50 60 70 80 90 1 METER CENTIMETERS
FIG. 21. VERTICAL SECTION THROUGH CREST
0 1 2 3 4 5 6 INCHES
0 2 4 6 8 10 12 14 CENTIMETERS
FIG. 16. PLAN
GATE
CREST
PIER
BRASS PIPE TO HOOK GAUGE
HOOK GAUGES
SCREEN
BRUSH
SUDBURY RIVER AQUEDUCT
SCALE 0 1 2 3 4 5 6 7 FEET
SCALE 0 1 2 METERS
A
B
FIG. 17 SECTION ON A-B
BRUSH SCREEN
FIG. 18. HORIZONTAL SECTION THROUGH STRUCTURE OF TERMINAL CHAMBER
WEST WEIR
EAST WEIR
CLAY PUDDLING
SCALE 0 1 2 3 4 5 6 7 8 9 10 11 12 13 FEET
SCALE 0 1 2 3 METERS

duct, the experiments were begun at the siphon chamber and were continued until 2.40 P. M.

In order to ascertain the time required for the water to pass from the weir at the siphon to the weirs at the terminal chamber, floats were put into the water below the siphon weir at regular intervals. They were received at the terminal chamber at intervals corresponding almost exactly. Accordingly, observations at the terminal weirs were regularly begun when the proper time had elapsed after the beginning of the siphon observations, and they were continued for the same length of time as at the siphon.

During the comparative observations care was taken not to disturb the gates at the head of the aqueduct, for it has been found that while it takes more than eight hours for the water to travel the length of the aqueduct, the slightest disturbance at the head gates sends an advance wave along the aqueduct with great rapidity. If one of these waves starts from the head of the aqueduct at the same time that a float is put into the aqueduct at the siphon chamber, the wave overtakes the float before it arrives at the terminal chamber. That is to say, the wave travels through 17 miles of aqueduct in less time than the water passes along the last 5 miles.

Table No. 5 shows the comparisons made between the weirs at siphon and terminal gate-chambers.

The mean ratio of the discharge (Q) at the siphon weir to the discharge (Q) at the terminal chamber was 0.9886, and all the discharges found at the terminal chamber were reduced to the standard of the siphon weir by the use of that ratio.

Leakage into the Aqueduct.—The leakage into the 5 miles of aqueduct was determined by weir measurements, and was found to amount to 0.558 cu. ft. per second.

Levels.—The zeros of the gauges were set by very careful spirit leveling, the probable error by least squares being 0.002 ft. in the distance of 1 800 ft.

Water Level Observations.—As a check on the spirit leveling the 48-in. pipes were alternately filled with water and shut off, so that the flow was stopped, and the elevations of the water surfaces were observed at both ends by reading the gauges. The readings at the two ends of the pipes were compared, and their differences were tabulated and plotted.

All the differences in the case of the tube piezometers and fully 75% in the case of the other piezometers were in the same direction, whence it was concluded that the zeros of the gauges had not been set at exactly the same level. Tables Nos. 6 and 7 give a summary of the comparison. It appears from Table No. 6, embracing the outside piezometers, that in the case of the north pipe, the zero of the scale at the west end was probably too low by 0.0019 ft., and that in the case of the south pipe it was too low by 0.0020 ft., relatively to the zeros at the east end.

From Table No. 7, it appears that the zeros of the west tube gauge piezometers were likewise too low by 0.0083 ft. in the case of the north pipe, and by 0.0065 ft. in the case of the south pipe. These gauges had not been set with as great care as the outside piezometer gauges.

By the method of least squares it was found that the difference in elevation between water surfaces at piezometer gauges could be determined from the means of all the water-level observations with a probable error much smaller than the probable error in the spirit leveling. This is owing to the very large number of readings of the water level. A close analysis of them convinced the author that their mean could be relied upon within 0.001 ft.

In the tables the values c_1 and c_3 have been calculated upon the basis of the spirit leveling. In order to substitute the relation determined by the water-level observations, it is necessary to make a slight correction. The values as corrected are given in the tables, and marked c_2 and c_4. For ordinary velocities their difference from c_1 and c_3 is insignificant. The diagrams (Figs. 22 and 23) have been plotted, using the values of c_2 and c_4.

Oscillations in the Isolated Pipes.—The comparisons of the water-level observations just mentioned were noticeably different for different times. To account for these differences, it was suggested that the effect of the wind might cause a difference in the pressure of the atmosphere between the two ends of the pipe. If atmospheric differences existed, it was evident that a correction on that account should be applied to the observed losses of head in the flowing pipe.

It had been found that the two pairs of piezometers at the same section gave the same reading, and that, therefore, the use of either

pair alone was sufficient. Accordingly, while the tube on one side of the gauge continued to be used for a pair of piezometers in the flowing pipe, the tube on the other side of the gauge was connected with the still pipe. Observations on the latter were continued during a large portion of the experimenting; and, as it was assumed that the same variations in atmospheric pressure were acting upon both pipes, it was intended that the results of each 100 minutes' observations of the flow should be corrected by an amount determined by the simultaneous observations on the still pipe. The comparative observations, however, did not afford a basis for making the corrections as contemplated.

There appeared to be no correspondence between the variations in the differences of water level and the variations in the velocity and direction of the wind. Moreover, for the same interval of time the difference of the readings on one set of gauges was often very different from that on the other set of gauges, and was opposite in direction in somewhat less than 25% of the observations of the outside piezometers.

The variations in question appeared to be due to an oscillating movement of the water in the pipe. The period of oscillation in the 48-in. pipe, if 1 800 ft. long, was found by calculation to be 33 seconds, neglecting the effects of friction. The observed period, being affected by friction, was from 60 to 70 seconds. The effect of the oscillations of the water in the pipe was shown on diagrams plotted from special readings made for the purpose on October 6th, 1894. It was seen from these plots that while an observer at one end of the pipe was reading the crest of the wave, the observer at the other end was reading the hollow, and that the period of oscillation was so nearly one minute that readings taken at intervals of two minutes did not give the mean position of the water surface; and it was at intervals of two minutes that the readings were taken during all of the observations in the tube piezometers, and on about half of the experiments on the other piezometers.

Correct results in such cases can only be obtained by taking the observations so often as to nullify the effect of the oscillations, which is often impracticable. The error was much smaller in the case of the readings taken every half minute, which almost eliminated the effect of the oscillations. The amplitude of the oscillations, as determined

in the piezometric readings, was, of course, modified by the throttling at the gauges. It was found that at the open end of the east end of the 48-in. pipe the amplitude was about 0.03 ft., while at the east piezometer it was 0.005 ft. If any correction were to be applied on account of the oscillation, it would, as above seen, be very small, and it would not be well worth making where there exists an uncertainty nearly as great arising from other causes.

It was found by experiment that readings of gauges similar to those used for the tube piezometers at the siphon are uncertain to the extent of 0.002 ft., probably due to variations in capillary attraction in the ½-in. glass tubes. The outside piezometer gauges had larger glass tubes. It further appeared from an actual test, continued throughout a whole day, that there is a liability to errors of observation in the case of the piezometric readings amounting to 0.001 ft., and to somewhat more than that in the case of the tube piezometers, which were unprovided with verniers. Four series of observations of 100 minutes each were made. There were four observers employed, two at each of the two piezometer gauges, and they changed ends several times during the day. One observer read the column on one side of the gauge, and the other, that on the other side. Readings were taken every half minute, the observers reading simultaneously for half the day, and alternately at quarter-minute intervals the other half day. The differences of one observer from the other on the mean of 100 minutes were 0.0003, 0.0006, 0.0003, 0.0003, 0.0006, 0.0001, 0.0013 and 0.0013.

The author has gone into this matter fully, not so much on account of its value in determining the coefficients, as for the purpose of showing the refinements to which these experiments were carried, and pointing out what perplexing sources of error are liable to be encountered in such work. In ordinary experiments to determine the losses of head it would be unnecessary to go to the expense of such fine apparatus.

Degree of Accuracy in Measurements and Results.—The approximate uniformity of the piezometric readings can be seen by a study of the tables, which give the highest and lowest readings in the different 20-minute intervals, with the range, which is generally less than 0.02 ft.

The length of the pipe has probably been obtained with a precis-

ion of 1 in 50 000, which is 0.002%, and would equal about 0.03 ft. in a length of 1 600 ft.

The accuracy of the leveling and of the reading of the gauges has already been discussed. Upon these elements the accuracy of the determination of the loss of head depends.

The measurement of the discharge is probably liable to an error of 1%, and, as the velocities vary directly as the discharge, the velocity is liable to an error of 1%, or, say, from an error of 0.003 ft. per second for the smallest velocity to 0.07 ft. per second for the largest. The coefficient c is in error about 1% if the velocity is in error 1 per cent.

Formula for Weirs.—Much study was given to the question what formula to use in computing the discharge of the weirs, and it was finally decided to use the result of Bazin's experiments, as best fitting the case in hand. The formula is $Q = m\,L\,H\sqrt{2gH}$. The values of the coefficient m for each height of weir employed were tabulated from the results of Bazin's experiments and used in the computations.

Notation.—Quantities entering into the formulas are expressed as follows when English measures are used :

I = friction head in feet per foot of pipe.
v = mean velocity in feet per second.
R = hydraulic mean depth in feet.

Formula for Flow in Pipes.—For the want of any really satisfactory formula to express the law of flow in pipes generally, the familiar Chézy formula, $v = c\,(R\,I)^{\frac{1}{2}}$ has been made the basis of study for these experiments. In the pipes in question, the diameter being 4 ft., $R = 1$ and vanishes, so that the formula becomes $v = c\,I^{\frac{1}{2}}$. It being impossible to assign to c in that formula any one value which will fit all cases, the attempt has been made to find what different values of c will fit the various conditions experimented upon.

Simple Chézy Formula Sufficient Only for Particular Condition of Surface of Pipe.—In the tuberculated pipes experimented upon, a constant value of 108 for c, making the formula $v = 108\,I^{\frac{1}{2}}$, fits the experiments well for all the heads. The form $v = 100\,I^{\frac{1}{2}}$ is still easier to remember, and is excellent to express the flow through 48-in. pipe

slightly more tuberculated, say with 20 years' service. For metric measures these formulas become $v = 59.63\ (R\,I)^{\frac{1}{2}}$ and $v = 55.21\ (R\,I)^{\frac{1}{2}}$, respectively. To fit the experiments on the clean pipe closely, however, different values of c are required for different heads.

The Substitution of Kutter's Coefficients Adapts the Formula to Different Cases and Particularly to Clean Pipe.—In substituting values for c the method of Kutter's formula is admirable in that it takes account separately of the different elements that modify the coefficient, to wit, the hydraulic mean depth, the loss of head and the condition of the surface as to roughness. The last is allowed for by the introduction of a quantity, represented by n, called the coefficient of roughness. This is of especial importance, it being found that the rusting of the interior of a pipe diminishes its capacity very much, as already stated in the case of the Rosemary siphon. In designing works a large allowance is necessarily made for this, if they are expected to last for many years. The coefficient c according to Kutter's formula fits well the experiments on the clean pipe, taking $n = 0.011$. The formula then becomes

$$v = \frac{206.2\ I + 0.00281}{1.458\ I + 0.000031} \times I^{\frac{1}{2}}.$$

To fit the experiments on the tuberculated pipe as nearly as possible with Kutter's formula, n should be taken about 0.014, which is about the same value that he uses for brickwork. The formula then becomes

$$v = \frac{171\ I + 0.00281}{1.582\ I + 0.0000393} \times I^{\frac{1}{2}}.$$

To fit what the author believes would be the condition of a pipe badly tuberculated, say by fifty years' service, n in Kutter's formula should be taken as large as 0.0157, and the formula would be

$$v = \frac{157\ I + 0.00281}{1.653\ I + 0.0000441} \times I^{\frac{1}{2}}.$$

This formula is not based on experiment.

In the case of the tuberculated pipes the Kutter formula does not fit the low heads, say for velocities of less than 1 ft. per second. At low velocities the loss of head due to friction is very small and difficult to measure with accuracy relatively to the magnitudes involved.

Hence the probable error is relatively large, and at the best the precise form of the curve which would graphically express c is doubtful for very low heads. As n is the coefficient of roughness, it was intended that the same value should be assigned to it in a pipe in any one given state, irrespectively of whether the velocity of the water running through it is great or small; an idea of the imperfection of the Kutter formula if applied to these experiments may therefore be obtained from the statement that instead of 0.014, as above mentioned for the value of n for considerable velocities, it would be necessary in the case of the Rosemary south pipe, in order to fit the Kutter formula for the velocity of 1.2 ft. per second, to make n about 0.013, and for the velocity of 0.7 ft. per second, about 0.012.

Formula Chosen for the Present Investigation.—It was found that for the author's experiments an exponential formula would fit the case of the tuberculated pipe much better, and of the clean pipe somewhat better than Kutter's formula.

The desired formula was obtained by the method of logarithmic homologues described by Professor Reynolds in the *Philosophical Transactions* of the Royal Society, London, 1883. The logarithms of I were plotted as abscissas, and the logarithms of v as ordinates; and from drawing straight lines, coinciding as nearly as possible with the plotted points on these logarithmic diagrams, exponents were obtained from which the following formulas are derived:

	North Pipe.		South Pipe.
	Cleaned.	Tuberculated.	Tuberculated.
v	$166\ I^{\frac{1}{1.91}}$	$99.5\ I^{\frac{1}{2.03}}$	$105.4\ I^{\frac{1}{2.02}}$
v, feet	$170\ I^{\frac{1}{1.9}}$	$108\ I^{\frac{1}{2}}$	
v, meters	$93.86\ I^{\frac{1}{1.9}}\ R^{\frac{1}{2}}$	$59.63\ I^{\frac{1}{2}}\ R^{\frac{1}{2}}$	

The expressions in the last two lines are obtained by rounding off figures for greater simplicity; they might be used as the basis for computing tables if desired.

The curves plotted on the diagrams of c, Figs. 22 and 23, are based on calculation from formulas given in the first line of the preceding table.

Experiments of Mr. Stearns.—The experiments recorded in this paper possess an additional value from the fact that the coefficients for the same pipes when new were determined by F. P. Stearns, M. Am. Soc. C. E., as communicated to this Society in his paper read October 1st, 1884.*

By the three experiments which Mr. Stearns regarded as trustworthy the coefficient c in the Chézy formula, $v = c\,(R\,I)^{\frac{1}{2}}$ had values as shown in the second column of the table below, for the velocities given in the first column respectively. The corresponding coefficients as determined by the experiments of 1894–95 in the clean pipe (tubercles removed) calculated by the formula $c = 131.88\ v^{0.045}$ (the same value used in formula $166\ I^{\frac{1}{1.91}}$), are given in the third column.

VELOCITY.	c, 1880.	c, 1894–95.
3.738	140.14	139.94
4.965	142.11	141.74
6 195	144.09	143.16

Conclusion.—In referring to the number of years of service of pipes as indicative of the condition of the interior surface, it should be observed that in other localities the effect of use may not be the same as on the Boston Water-Works. Many waters, for example, containing lime produce a smooth white coating inside the pipes. It is greatly to be desired that more accurate observations and experiments should be made as to how the frictional loss of head is affected by such coating, and whether it becomes further modified by longer periods of service.

The results herewith presented led the author to the conclusion that piezometric gauges laid upon the bottom of a pipe and those screwed into the sides give equally accurate results and that these piezometers when properly arranged can be depended upon as certainly as can other hydraulic appliances of precision. The author is particular in calling attention to this fact on account of slurs

* See the *Transactions* of the American Society of Civil Engineers, Vol. xiv, p. 1.

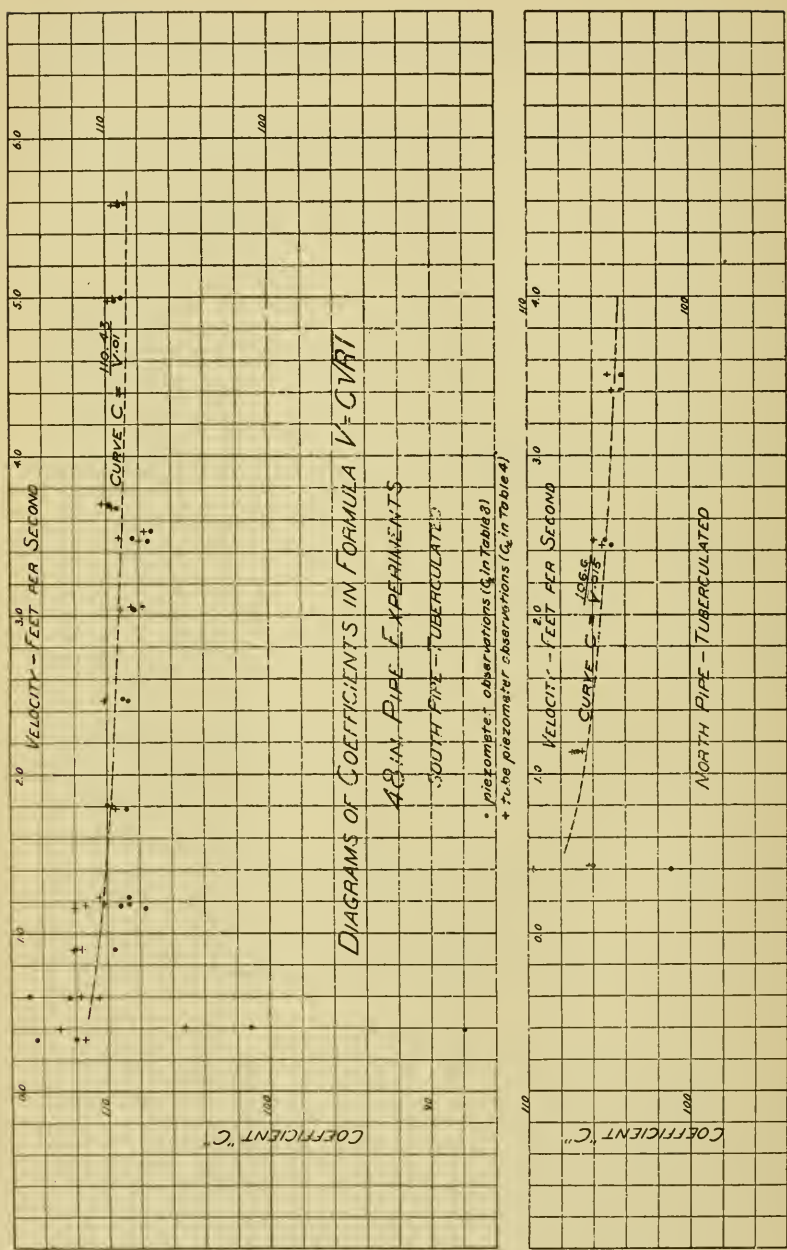

DIAGRAMS OF COEFFICIENTS IN FORMULA V=CVRI
48 IN. PIPE EXPERIMENTS
SOUTH PIPE - TUBERCULATED
• piezometer observations (C1 in Table 3)
+ tube piezometer observations (C4 in Table 4)
CURVE C = 110.43 / V.01
VELOCITY - FEET PER SECOND
COEFFICIENT "C"
NORTH PIPE - TUBERCULATED
CURVE C = 106.6 / V.015

FIG. 22.

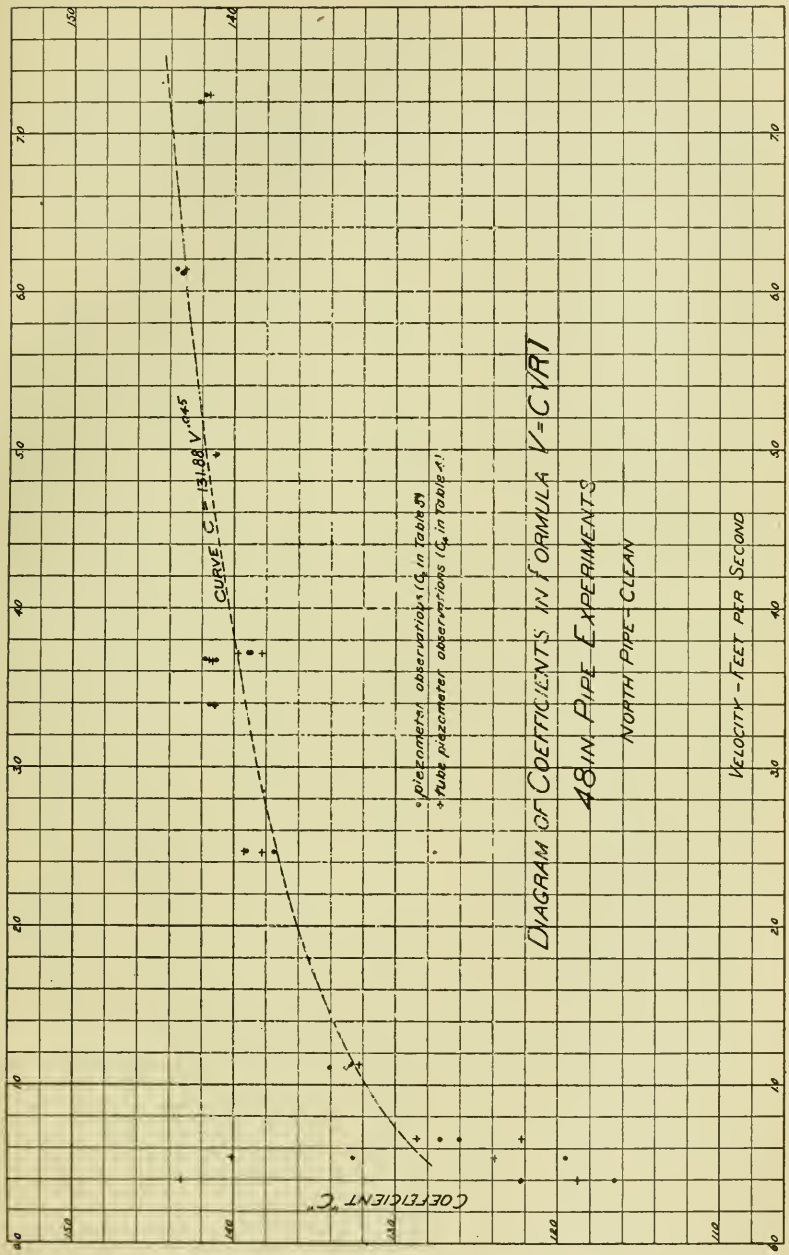


DIAGRAM OF COEFFICIENTS IN FORMULA V=CVRI
48 IN. PIPE EXPERIMENTS
NORTH PIPE - CLEAN
VELOCITY - FEET PER SECOND
COEFFICIENT "C"
CURVE C = 131.88 V^.045
• piezometer observations (C₃ in Table 5)
+ tube piezometer observations (C₄ in Table 4)

FIG. 23.

that have been cast by some hydraulicians upon piezometric observations.

In conclusion the author desires to express his obligations to Messrs. William E. Foss, Frank S. Hart and F. F. Moore, principal assistants in these experiments, to whose skill and zeal the success of the work is largely due, and to Fred. Brooks, M. Am. Soc. C. E., for advice in the preparation of this paper.

TABLE No. 1.—WEIR OBSERVATIONS (Terminal Chamber).

Using Bazin's values of m in formula $Q = m\ L\ H \sqrt{2\ g\ H}$; $g = 32.17$.

Experiments 216 or XLIV to LI were on north pipe clean ; LI to LVI, inclusive, on south pipe tuberculated (see note to Table No. 2).

1	2	3		4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22
				East Weir.								West Weir.										
				Height of water on weir.								Height of water on weir.										
Date.	Number of experiment.	Time of beginning.	Time of ending.	No. observations.	Highest. Feet.	Lowest. Feet.	Range. Feet.	Mean. H. Feet.	Mean length of weir. L. Feet.	Coefficient from table. m.	Quantity in cubic feet, per second. Q.	No. observations.	Highest. Feet.	Lowest. Feet.	Range. Feet.	Mean. H. Feet.	Mean length of weir. L. Feet.	Coefficient from table. m.	Quantity in cubic feet per second. Q.	Total quantity. Column 11 plus column 19.	Total quantity, less 0.558 cubic feet per second, leakage.	Column 21. x 0.9886 to reduce to siphon weir.
1894.																						
Dec. 24th.	216	1.00	1.20	10	1.5081	1.5040	0.0041	1.5057	6.3244	0.4322	40.509	11	1.5265	1.5214	0.0051	1.5228	5.8403	0.4324	38.065	78.574	78.016	77.127
" 24th.	217	1.20	1.40	10	1.5070	1.5029	0.0041	1.5052	6.3244	0.4322	40.489	11	1.5262	1.5219	0.0043	1.5240	5.8403	0.4324	38.110	78.599	78.041	77.151
" 24th.	218	1.40	2.00	10	1.5072	1.5032	0.0040	1.5055	6.3244	0.4322	40.501	11	1.5272	1.5203	0.0069	1.5243	5.8403	0.4324	38.121	78.622	78.064	77.174
" 24th.	219	2.00	2.20	10	1.5090	1.5035	0.0055	1.5068	6.3244	0.4322	40.553	11	1.5275	1.5203	0.0072	1.5236	5.8403	0.4324	38.095	78.648	78.090	77.200
" 24th.	220	2 20	2.40	10	1.5083	1.5034	0.0049	1.5062	6.3244	0 4322	40.529	11	1.5250	1.5210	0.0040	1.5231	5.8403	9.4324	38.076	78.605	78.047	77.157
.	(XLIV)	1.00	2.40	50	1.5090	1.5029	0.0061	1.5059	6.3244	0.4322	40.517	51	1.5275	1.5203	0.0072	1.5236	5.8403	0.4324	38.095	78.612	78.054	77.164
Dec. 24th.	(XLV)	4.20	6.00	50	1.5065	1.4989	0.0076	1.5032	6.3244	0.4322	40.408	51	1.5240	1.5171	0.0069	1.5207	5.8403	0.4324	37.986	78.394	77.836	76.949
1895.																						
Jan. 10th.	(XLVI)	2.40	4.20	51	1.0903	1.0854	0.0049	1.0876	6.3206	0.4285	24.641	50	1.1061	1.1001	0.0060	1.1029	5.8367	0.4286	23.242	47.883	47.325	46.785
" 10th.	(XLVII)	6.00	7.40	51	1.0940	1.0841	0.0099	1.0875	6.3206	0.4285	24.637	50	1.1091	1.0996	0.0095	1.1030	5.8367	0.4287	23.250	47.887	47.329	46.789
" 11th.	(XLVIII)	2.30	4.10	51	1.3168	1 3073	0.0095	1.3133	6.3200	0.4304	32.838	50	1.3337	1.3259	0.0078	1.3304	5.8364	0.4306	30.934	63.772	63.214	62.493
" 16th.	(XLIX)	2.00	3.40	51	1.6728	1.6660	0 0068	1.6692	6.3185	0.4338	47.414	50	1.6956	1.6866	0.0090	1.6914	5.8354	0.4340	44.686	92.100	91.542	90.498
" 16th.	(L)	5.20	7.00	51	1.6790	1.6718	0.0072	1.6761	6.3184	0.4339	47.719	50	1.7014	1.6941	0.0083	1.6972	5.8354	0.4341	44.926	92.645	92.087	91.037
" 23d .	(LI)	2.40	4.20	51	1.0770	1.0715	0.0055	1.0744	6.3207	0.4284	24.188	50	1.1026	1·0965	0.0061	1.0995	5.8367	0.4286	23.134	47.322	46.764	46.231
" 23d .	(LII)	6.00	7.40	51	1 0864	1.0740	0.0124	1.0792	6.3207	0.4285	24.356	50	1.1111	1.1003	0.0108	1.1041	5.8367	0.4287	23.285	47.641	47.083	46.546
" 24th.	(LIII)	2.30	4.10	51	1.3120	1.3030	0.0090	1.3077	6.3200	0.4303	32.621	50	1.3436	1.3351	0.0085	1.3391	5.8364	0.4306	31.238	63.859	63.301	62 579
" 24th.	(LIV)	5.50	7.30	51	1.3174	1.3065	0.0109	1.3117	6.3200	0.4304	32.778	50	1.3461	1.3379	0.0082	1.3428	5.8364	0.4307	31.375	64.153	63.595	62.870
" 25th.	(LV)	2 20	4.00	51	1.4120	1.4048	0.0072	1.4078	6.3197	0.4313	36.520	50	1.4444	1.4385	0.0059	1.4409	5.8362	0.4316	34.946	71.466	70.908	70.100
" 25th.	(LVI)	5.40	7.20	51	1.4152	1.4045	0.0107	1.4099	6.3197	0.4313	36.602	50	1.4472	1.4391	0.0081	1.4429	5.8362	0.4316	35.019	71.621	71.063	70.253

NOTE.—The temperatures during the experiments were as follows: 216, 36.7°; XLV, 36.8; XLVI, 37°; XLVII, 37°; XLVIII, 36.7°; XLIX, 35.5°; L, 35.3°; LI, 35.5°; LIII, 35.1°; LV, 34.9°.

TABLE No. 2. WEIR OBSERVATIONS (Siphon).*

Using Bazin's values of m in formula $Q = m\ L\ H \sqrt{2gH}$; $g = 32.17$.

Experiments 1 or I to XXII were on the tuberculated south pipe, XXII to XXX on the north pipe tuberculated, and XXX to XLIV on the north pipe clean.

1	2	3		4	5	6	7	8	9	10	11	12
				Height of Water on Weir.								
Date.	Number of experiment.	Time of beginning.	Time of ending.	Number of observations.	Highest. Feet.	Lowest. Feet.	Range. Feet.	Mean. H. Feet.	Mean length of weir. Feet. L.	Coefficient from table. m.	Quantity in cubic feet per second. Q.	Temperature of water, Fahrenheit.
1894. September 4th. P.M..	1	1:20	1:40	10	1.8438	1.8277	0.0161	1.8347	4.9707	0.4421	43.805	64.2°
	2	1:40	2:00	11	1.8420	1.8302	0.0118	1.8368	4.9707	0.4422	43.890	
	3	2:00	2:20	11	1.8399	1.8332	0.0067	1.8368	4.9707	0.4422	43.890	
	4	2:20	2:40	11	1.8385	1.8356	0.0029	1.8372	4.9707	0.4422	43.905	
	5	2:40	3:00	11	1.8390	1.8358	0.0032	1.8375	4.9707	0.4422	43.916	
	(I)	1:20	3:00	50	1.8438	1.8277	0.0161	1.8366	4.9707	0.4422	43.883	
1894. September 5th. A.M..	(II)	10:00	11:40	51	1.4778	1.4700	0.0078	1.4740	4.9712	0.4373	31.205	66.2°
" 5th. P.M..	(III)	1:20	3:00	51	1.4710	1.4640	0.0070	1.4674	4.9712	0.4372	30.989	
" 6th. A.M..	(IV)	10:00	11:40	51	0.9143	0.8990	0.0153	0.9050	4.9724	0.4303	14.776	
" 6th. P.M..	(V)	1:20	3:00	51	0.9029	0.8951	0.0078	0.8990	4.9725	0.4303	14.629	
" 7th. A.M..	(VI)	10:00	11:40	51	0.4475	0.4380	0.0095	0.4431	4.9746	0.4311	5.074	65.1°
" 7th. P.M..	(VII)	1:20	3:00	51	0.4400	0.4333	0.0067	0.4368	4.9747	0.4312	4.967	65.1°
" 11th. A.M..	(VIII)	10:00	11:40	51	1.6973	1.6785	0.0188	1.6858	4.9709	0.4401	38.409	70.3°
" 11th. P.M..	(IX)	1:20	3:00	51	1.6832	1.6763	0.0069	1.6798	4.9709	0.4401	38.204	70.3°
" 12th. A.M..	(X)	10:00	11:40	51	1.2058	1.1966	0.0092	1.2005	4.9717	0.4337	22.750	68.8°
" 12th. P.M..	(XI)	1:20	3:00	51	1.2036	1.1830	0.0206	1.1930	4.9717	0.4336	22.532	
" 13th. A.M..	(XII)	10:00	11 40	51	0.7775	0.7593	0.0182	0.7614	4.9732	0.4290	11.370	67.7°
" 13th. P.M..	(XIII)	1:20	3:00	51	0.7618	0.7525	0.0093	0.7590	4.9732	0.4290	11.316	
" 14th. A.M..	(XIV)	10:00	11:40	51	0.5900	0.5769	0.0131	0.5823	4.9740	0.4291	7.607	67.6°
" 14th. P.M..	(XV)	1:20	3:00	51	0.5796	0.5725	0.0071	0.5756	4.9740	0.4292	7.478	

October	4th. A.M..	(XVI)	10:00	11.40	51	1.8404	1.8170	0.0234	1.8302	4.9688	0.4421	43.628	61.4°
"	4th. P.M..	(XVII)	1:20	3.00	51	1.8595	1.8406	0.0189	1.8510	4.9687	0.4424	44.403	
"	5th. A.M..	(XVIII)	10:00	11:40	51	0.9405	0.9263	0.0142	0.9312	4.9702	0.4306	15.426	61.8°
"	5th. P.M..	(XIX)	1:20	3:00	51	0.9174	0.9102	0.0072	0.9136	4.9702	0.4304	14.984	
"	6th. A.M..	(XX)	10:00	11:40	51	0.3922	0.3860	0.0062	0.3883	4.9728	0.4323	4.172	61.6°
"	6th. P.M..	(XXI)	1:20	3:00	51	0.4067	0.3777	0.0290	0.3845	4.9728	0.4324	4.112	
"	18th. A.M..	(XXII)	10.00	11:40	51	1.8172	1.8007	0.0165	1.8101	4.9688	0.4418	42.882	53.3°
"	18th. P.M..	(XXIII)	1:20	3:00	48	1.8564	1.8324	0.0240	1.8130	4.9687	0.4423	44.105	
"	19th. A.M..	(XXIV)	10 00	11:40	51	1.4732	1.4635	0.0097	1.4690	4.9692	0.4373	31.034	52.8°
"	19th. P.M..	(XXV)	1:20	3:00	51	1.4613	1.4545	0.0068	1.4575	4.9693	0.4371	30.657	
"	20th. A.M	(XXVI)	10:00	11:40	51	0.4568	0.4465	0.0103	0.4523	4.9724	0.4309	5.228	
"	20th. P.M..	(XXVII)	1:20	3:00	51	0.4445	0.4393	0.0052	0.4420	4.9724	0 4311	5.053	
"	30th. A M..	(XXVIII)	10:00	11:40	50	0.8897	0.8801	0.0096	0.8847	4.9703	0.4301	14.269	
"	30th. P.M..	(XXIX)	1:20	3:00	50	0.8894	0.8828	0.0066	0.8860	4.9703	0.4301	14.300	
November	30th. A M..	(XXX)	10:00	11:40	51	1.9035	1.8943	0.0092	1.8979	4.9693	0.4430	46.169	
"	30th. P.M..	(XXXI)	1:20	3:00	50	1.9053	1.8966	0.0087	1.9009	4.9693	0.4430	46.278	
December	1st. A.M..	(XXXII)	10:00	11:40	51	1.4751	1.4646	0.0105	1.4699	4.9700	1.4373	31.068	35.5°
"	1st. P.M..	(XXXIII)	1:20	3:00	51	1.4759	1.4641	0.0118	1.4714	4.9700	0.4373	31.115	
"	7th. A M..	(XXXIV)	10:00	11:40	51	0.8809	0.8672	0.0137	0.8731	4.9714	0.4300	13.989	36.5°
"	7th. P.M..	(XXXV)	1:20	3:00	51	0.8845	0.8765	0.0080	0.8808	4.9713	0.4301	14.177	
"	8th. A.M..	(XXXVI)	10:00	11:40	51	0.4391	0.4315	0.0076	0.4369	4.9735	0 4312	4.968	37.3°
"	8th. P.M..	(XXXVII)	1:20	3.00	51	0.4410	0.4325	0.0085	0.4387	4.9735	0.4312	4.998	
"	13th. A.M..	(XXXVIII)	10:00	11:40	51	1.8108	1.7878	0.0230	1.8009	4.9694	0.4417	42.551	36.5°
"	13th. P.M..	(XXXIX)	1:20	3:00	51	1.8113	1.7877	0.0236	1.8013	4.9694	0.4417	42.565	
"	14th. A.M..	(XL)	10:00	11:40	51	0.6225	0.6113	0 0112	0.6168	4.9727	0.4289	8.287	37.0°
"	14th. P.M..	(XLI)	1:20	3:00	51	0.6249	0.6071	0.0178	0.6148	4.9727	0 4289	8.247	
"	15th. A.M..	(XLII)	10:00	11:40	51	0.5483	0.5323	0.0160	0.5362	4.9730	0.4295	6.727	38.3°
"	15th. P.M..	(XLIII)	1:20	3:00	49	0.5430	0.5350	0.0080	0.5388	4.9730	0.4295	6.776	

* It was intended originally to print the tables in full, but on account of the space required they have been condensed. Each experiment, covering 20 minutes and embracing ten observations, was numbered as a distinct experiment with arabic figures. Five of these experiments, covering 100 minutes, were united and a mean obtained, forming a set of means designated by Roman numerals. The latter only have been printed, but with each table a sample of the whole method on which the table was formed is given to show the slight variations in the 20-minute experiments.

TABLE No. 3.—Piezometer Observations.

1	2	3		4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
				West Piezometer Readings.					East Piezometer Readings.											
Date.	Number of experiment.	Time of beginning.	Time of ending.	Number of observations.	Highest. Feet.	Lowest. Feet.	Range. Feet.	Mean. Feet.	Number of observations.	Highest. Feet.	Lowest. Feet.	Range. Feet.	Mean. Feet.	Mean total fall in 1 615.7 feet of pipe. Feet.	Fall per foot of pipe I_1. Feet.	Discharge from table 1, column 11; and table 2, column 22. Cubic feet per second.	Mean velocity of flow in pipe. v. Feet per second.	Coefficient in $V = c_1 \sqrt{RI_1}$. c_1.	Results from columns 15 and 18 corrected for water levels. I_2.	c_2.
South Pipe Tuberculated.—Length, 1 615.70 Ft.																				
1894. Sept. 4th, P.M..	1	1.20	1.40	21	6.548	6.541	0 007	6.5438	21	4.877	4.865	0.012	4.8695	1.6743	0.0010363	43.805	3.486	108.29		
	2	1.40	2.0	21	6.549	6.542	0.007	6.5439	21	4.873	4.860	0.013	4.8669	1.6770	0.0010379	43.890	3.493	108.41		
	3	2.0	2.20	21	6.552	6.544	0.008	6.5484	21	4.873	4.858	0.015	4.8650	1.6834	0.0010419	43.890	3.493	108.20		
	4	2.20	2.40	21	6.554	6.548	0.006	6.5513	21	4.876	4.867	0.009	4.8721	1.6792	0.0010393	43.905	3.494	108.38		
	5	2.40	3.0	21	6.557	6.552	0.005	6.5541	21	4.877	4.868	0.009	4.8728	1.6813	0.0010406	43.916	3.495	108.33		
	(I)	1.20	3.0	101	6.557	6.541	0.016	6.5483	101	4.877	4.858	0.019	4.8693	1.6790	0.0010392	43.883	3.492	108.33	0.0010383	108.38
1894. Sept. 5th, A.M..	(II)	10.0	11.40	101	5.347	5.338	0.009	5.3431	101	4.509	4.497	0.012	4.5023	0.8408	0.0005204	31.205	2.483	108.86	0.0005195	108.96
" 5th, P.M..	(III)	1.20	3.0	101	5.337	5.330	0.007	5.3340	101	4.507	4.497	0.010	4.5006	0.8334	0.0005158	30.989	2.466	108.58	0.0005149	108.68
" 6th, A.M..	(IV)	10.0	11.40	101	5.345	5.340	0.005	5.3418	101	5.159	5.146	0.013	5 1530	0.1888	0.0001169	14.776	1.176	108.77	0.0001160	109.20
" 6th, P.M..	(V)	1.20	3.0	101	5.345	5.341	0.004	5.3432	101	5.159	5.149	0.010	5.1528	0.1904	0.0001178	14.629	1.164	107.24	0.0001169	107.66
" 7th, A.M..	(VI)	10.0	11.40	101	6.056	6.048	0.008	6.0513	101	6.026	6.023	0.003	6.0241	0.0272	0.0000168	5.074	0.404	98.40	0 0000159	101.23
" 7th, P.M..	(VII)	1.20	3.0	51	6.065	6.061	0.004	6.0617	51	6.031	6.026	0.005	6.0276	0.0341	0.0000211	4.967	0.395	86.04	0.0000202	88.00
" 11th, A.M..	(VIII)	10.0	11.40	51	6.026	6.003	0.023	6.0141	51	4.723	4.704	0.019	4.7133	1.3008	0.0008051	38.409	3.057	107.72	0.0008042	107.78
" 11th, P.M..	(IX)	1.20	3.0	51	5.998	5.975	0.023	5.9849	51	4.717	4.702	0.015	4.7108	1.2741	0.0007886	38.204	3.040	108.26	0.0007877	108.32
" 12th, A.M..	(X)	10.0	11.40	51	5.872	5.861	0.011	5.8681	51	5.438	5.419	0.019	5.4287	0.4394	0.0002720	22.750	1.810	109.78	0.0002711	109.97
" 12th, P.M..	(XI)	1.20	3.0	51	5.868	5.862	0.006	5.8611	51	5.433	5.414	0.019	5.4238	0.4404	0.0002726	22.532	1.793	108.60	0.0002717	108.79
" 13th, A.M..	(XII)	10.0	11.40	51	6.395	6.388	0.007	6.3921	51	6.287	6.284	0.003	6.2852	0.1069	0.0000662	11.370	0.905	111.23	0.0000653	112.02
" 13th, P.M..	(XIII)	1.20	3.0	51	6.398	6.391	0.007	6.3950	51	6.287	6.282	0.005	6.2844	0.1106	0.0000685	11.316	0.901	108.84	0.0000676	109.59
" 14th, A.M..	(XIV)	10.0	11.40	51	6.152	6.144	0.008	6.1484	51	6.106	6.100	0.006	6.1020	0.0464	0.0000287	7.607	0.605	112.96	0.0000278	114.83
" 14th, P.M..	(XV)	1.20	3.0	51	6.148	6.142	0.006	6.1453	51	6.101	6.097	0.004	6.0985	0.0468	0.0000290	7.478	0.595	110.57	0.0000281	112.39
Oct. 4th, A.M..	(XVI)	10.0	11.40	51	6.574	6.515	0.059	6.5465	51	4.868	4.848	0.020	4.8580	1.6885	0.0010451	43.628	3.472	107.39	0.0010442	107.44
" 4th, P.M..	(XVII)	1.20	3.0	51	6.655	6.617	0.038	6.6374	51	4 892	4.869	0.023	4.8815	1.7559	0.0010868	44.403	3.533	107.18	0.0010859	107.23

Oct. 5th, A.M..	(XVIII)	10.0	11.40	51	5.407	5.392	0.015	5.4008	50	5.204	5.184	0.020	5.1932	0.2076	0.0001285	15.426	1.228	108.30	0.0001276	108.69
" 5th, P.M..	(XIX)	1.20	3.0	51	5.382	5.372	0.010	5.3762	51	5.191	5.174	0.017	5.1803	0.1959	0.0001212	14.984	1.192	108.29	0.0001203	108.71
" 6th, A.M..	(XX)	10.0	11.40	51	6.026	6.017	0.009	6.0205	51	6.012	6.000	0.012	6.0054	0.0151	0.0000093	4.172	0.332	108.61	0.0000084	114 44
" 6th, P.M..	(XXI)	1.20	3.0	51	6.023	6.009	0.014	6.0129	51	6.005	5.983	0.022	5.9976	0.0153	0.0000095	4.112	0.327	106.34	0.0000086	111.97
North Pipe Tuberculated.—Length, 1 615.70 Ft.																					
" 18th, A.M..	(XXII)	10.0	11.40	51	6.645	6.626	0.019	6.6339	51	4.915	4.883	0.032	4.8996	1.7343	0.0010734	42.882	3.412	104.15	0.0010717	104.23
" 18th, P M..	(XXIII)	1.20	3.0	51	6.802	6.739	0.063	6.7700	51	4.953	4.919	0.034	4.9345	1.8355	0.0011360	44.105	3.510	104.13	0.0011343	104.21
" 19th, A.M..	(XXIV)	10.0	11.40	396	5.442	5.415	0.027	5.4254	211	4.554	4.516	0.038	4.5316	0.8938	0.0005532	31.034	2.470	105.00	0.0005515	105.16
" 19th, P.M..	(XXV)	1.20	3.0	397	5 409	5.399	0.010	5.4033	199	4.532	4.519	0.013	4.5249	0.8784	0.0005437	30.657	2.440	104.63	0.0005420	104.79
" 20th, A.M..	(XXVI)	10.0	11 40	201	5.956	5.953	0.003	5.9541	201	5.929	5.924	0.005	5.9265	0.0276	0.0000171	5.228	0.416	100 66	0.0000154	105.98
" 20th, P.M..	(XXVII)	1.20	3.0	201	5.954	5 940	0.014	5.9507	201	5.925	5.921	0.004	5.9225	0.0282	0.0000175	5.053	0.402	96.24	0.0000158	101.21
" 30th, A.M..	(XXVIII)	10.0	11.40	201	5.329	5 322	0.007	5.3258	201	5.150	5.129	0.021	5.1416	0 1842	0.0001140	14.269	1.135	106.34	0.0001123	107.13
" 30th, P.M..	(XXIX)	1.20	3.0	201	5.332	5.328	0.004	5.3299	201	5.151	5 139	0.012	5.1443	0.1856	0.0001149	14.300	1.138	106.18	0.0001132	106.96
North Pipe Clean.—Length, 1 615.70 Ft.																					
Nov. 30th, A.M..	(XXX)	10.0	11.40	51	6.112	6.081	0.031	6.0971	51	5.012	4.987	0.025	4.9991	1.0980	0.0006796	46.169	3 674	140.93	0.0006779	141.10
" 30th, P.M..	(XXXI)	1 20	3.0	197	6.106	6.088	0.018	6.0966	201	5.016	4.991	0.025	5.0035	1.0931	0.0006765	46.278	3.683	141.59	0.0006748	141.77
Dec. 1st, A.M..	(XXXII)	10 0	11.40	201	5.072	5.047	0.025	5.0588	199	4.543	4.524	0.019	4.5335	0.5253	0.0003251	31.068	2.472	137.11	0.0003234	137.46
" 1st, P.M..	(XXXIII)	1.20	3.0	201	5.056	5.042	0.014	5.0487	196	4.545	4 523	0.022	4.5348	0.5139	0.0003181	31.115	2.476	138.84	0.0003164	139.21
" 7th, A.M..	(XXXIV)	10.0	11.40	201	6.523	6 510	0.013	6.5177	198	6.411	6.393	0.018	6.4036	0.1141	0.0000706	13.989	1.113	132.47	0.0000689	134 07
" 7th, P.M..	(XXXV)	1.20	3.0	201	6.535	6.522	0.013	6.5288	198	6.416	6.403	0.013	6.4094	0.1194	0.0000739	14.177	1.128	131.24	0 0000722	132.75
" 8th, A.M..	(XXXVI)	10.0	11.40	201	6 014	6.009	0.005	6.0118	201	5.993	5.989	0.004	5.9905	0.0213	0.0000132	4.968	0.395	108.88	0.0000115	116.51
" 8th, P.M..	(XXXVII)	1.20	3.0	197	6.017	6.011	0.006	6.0135	201	5.996	5.990	0.006	5.9937	0.0198	0.0000123	4.998	0.398	113.62	0.0000106	122 26
" 13th, A.M..	(XXXVIII)	10.0	11.40	201	5.835	5.809	0.026	5.8200	201	4 907	4.874	0.033	4.8885	0.9315	0.0005765	42.551	3.386	141.02	0.0005748	141.22
" 13th, P.M..	(XXXIX)	1.20	3.0	201	5.833	5.799	0.034	5.8141	201	4.909	4 871	0.038	4.8889	0.9252	0.0005726	42 565	3.387	141 55	0.0005709	141.76
" 14th, A.M..	(XL)	10.0	11.40	201	6.182	6.169	0.013	6.1734	199	6.137	6.123	0 014	6.1273	0.0461	0.0000285	8.287	0.659	123.46	0.0000268	127.24
" 14th, P M..	(XLI)	1 20	3.0	194	6.175	6.166	0.009	6.1711	201	6.131	6 121	0.010	6.1246	0.0465	0.0000288	8.247	0.656	122.33	0.0000271	126.04
" 15th, A.M..	(XLII)	10.0	11.40	201	6.097	6.085	0.012	6.0900	200	6.060	6.052	0.008	6.0549	0.0351	0.0000217	6.727	0.535	114.85	0 0000200	119.54
" 15th, P.M..	(XLIII)	1.20	3.0	196	6.096	6.085	0.011	6.0913	201	6.064	6.060	0.004	6.0619	0.0294	0.0000182	6.776	0.539	126.40	0.0000165	132.64
" 24th, A.M..	(XLIV)	10.0	11.40	188	6.002	5.932	0.070	5.9673	192	3.090	2.944	0.146	3.0106	2.9567	0.0018300	77.164	6 141	143.54	0.0018283	143.61
" 24th, P.M..	(XLV)	1.20	3 0	201	5.997	5.922	0.075	5.9604	196	3.089	2.912	0.177	3.0052	2.9552	0.0018291	76.949	6.123	143.18	0.0018274	143.25
1895.																					
Jan. 10th, A.M..	(XLVI)	10.0	11.40	187	5.953	5.930	0.023	5.9393	185	4.831	4.730	0.101	4.7786	1.1607	0.0007184	46.785	3.723	138.91	0.0007167	139.07
" 10th, P.M..	(XLVII)	1.20	3.0	186	5.960	5.930	0.030	5.9466	235	4.845	4.726	0.119	4.7834	1.1632	0.0007199	46.789	3.723	138.77	0.0007182	138.93
" 11th, A.M..	(XLVIII)	10.0	11.40	170	7.207	7.159	0.048	7.1835	186	5.217	5.139	0 078	5.1751	2.0084	0.0012431	62.493	4.973	141.05	0.0012414	141.14
" 16th, A.M..	(XLIX)	10 0	11.40	182	7.631	7.556	0.075	7.6012	191	3.477	3.439	0.038	3.4547	4.1465	0.0025664	90.498	7.202	142.16	0.0025647	142.21
" 16th, P.M..	(L)	1.20	3.0	182	7.746	7.680	0.066	7.7088	188	3.516	3.464	0.052	3.4902	4.2186	0.0026110	91.037	7.245	141.78	0.0026093	141.83
South Pipe Tuberculated.—Length, 1 615.70 Ft.																					
" 23d, A.M..	(LI)	10.0	11.40	186	6.015	5.990	0.025	6.0013	186	4.226	4.148	0.078	4.1742	1.8271	0.0011308	46.231	3.679	109.40	0.0011299	109.44
" 23d, P.M..	(LII)	1.20	3.0	180	6.037	6.010	0.027	6.0252	186	4.234	4.157	0.077	4.1847	1.8405	0.0011391	46.546	3.704	109.75	0.0011382	109.79
" 24th, A.M..	(LIII)	10.0	11.40	186	6.652	6.613	0.039	6.6323	178	3.368	3.225	0.143	3.2902	3.3421	0.0020685	62.579	4.980	109.49	0.0020676	109.51
" 24th, P.M..	(LIV)	1.20	3.0	185	6.717	6.689	0.028	6.7021	187	3.399	3.252	0.147	3.3017	3.4004	0.0021046	62.870	5.003	109.06	0.0021037	109.08
" 25th, A.M..	(LV)	10.0	11.40	170	6.873	6.844	0.029	6.8574	187	2.686	2.625	0.061	2.6378	4.2196	0.0026116	70.100	5.578	109.16	0.0026107	109.18
" 25th, P.M..	(LVI)	1.20	3.0	157	6.921	6.895	0.026	6.9098	187	2.662	2.634	0.028	2.6495	4.2603	0.0026368	70.253	5.591	108.87	0.0026359	108.89

TABLE No. 4.—Tube Piezometer Observations.

1	2	3		4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
				West Tube Piezometer Readings.					East Tube Piezometer Readings.									Results from cols. 15 and 17 corrected for water levels.	
Date.	Number of experiment.	Time of beginning.	Time of ending.	Number of observations.	Highest. Feet.	Lowest. Feet.	Range. Feet.	Mean. Feet.	Number of observations.	Highest Feet.	Lowest. Feet.	Range. Feet.	Mean. Feet.	Mean total fall in pipe. Feet.	Fall per foot of pipe. I_3. Feet.	Mean velocity of flow in pipe. From table 3, col. 17. v Feet per second.	Coefficient in $v = c_3\sqrt{RI_3}$. c_3.	I_4.	c_4.
South Pipe Tuberculated.—Length, 1 747.96 Ft.																			
1894. Sept. 4th, P.M..	1	1.20	1.40	11	6.639	6.631	0.008	6.6345	11	4.855	4.849	0.006	4.8508	1.7837	0.0010204	3.486	109.12		
	2	1.40	2.0	11	6.640	6.632	0.008	6.6357	11	4.850	4.845	0.005	4.8475	1.7882	0.0010230	3.493	109.20		
	3	2.0	2.20	11	6.643	6.635	0.008	6.6384	11	4.851	4.845	0.006	4.8493	1.7891	0.0010235	3.493	109.17		
	4	2.20	2.40	11	6.643	6.637	0.006	6.6420	11	4.850	4.848	0.002	4.8480	1.7940	0.0010263	3.494	109.06		
	5	2.40	3.0	11	6.645	6.638	0.007	6.6428	11	4.848	4.846	0.002	4.8470	1.7958	0.0010274	3.495	109.03		
	(I)	1.20	3.0	51	6.645	6.631	0.014	6.6387	51	4.855	4.845	0.010	4.8485	1.7902	0.0010242	3.492	109.12	0.0010205	109.32
1894. Sept. 5th, A.M..	(II)	10.0	11.40	51	5.398	5.383	0.015	5.3898	51	4.505	4.483	0.022	4.4931	0.8967	0.0005130	2.483	109.64	0.0005093	110.04
" 5th, P.M..	(III)	1.20	3.0	51	5.375	5.368	0.007	5.3710	51	4.491	4.486	0.005	4.4885	0.8825	0.0005049	2.466	109.75	0.0005012	110.16
" 6th, A.M	(IV)	10.0	11.40	51	5.351	5.345	0.006	5.3477	51	5.155	5.140	0.015	5.1464	0.2013	0.0001152	1.176	109.57	0.0001115	111.38
" 6th, P.M..	(V)	1.20	3.0	51	5.341	5.338	0.003	5.3394	51	5.150	5.140	0.010	5.1443	0.1951	0.0001116	1.164	110.19	0.0001079	112.07
" 7th, A.M..	(VI)	10.0	11.40	51	6.053	6.049	0 004	6.0514	51	6.023	6.016	0.007	6.0192	0.0322	0.0000184	0.404	94.07	0.0000147	105.30
" 7th, P.M..	(VII)	1.20	3.0	51	6.043	6.041	0.002	6.0418	51	6.016	6.013	0.003	6.0139	0.0279	0.0000160	0.395	98.94	0.0000123	112.97
" 11th, A.M..	(VIII)	10.0	11.40	85	6.109	6.085	0.026	6.0964	51	4.715	4.696	0.019	4.7035	1.3929	0.0007969	3.057	108.28	0.0007932	108.53
" 11th, P.M..	(IX)	1.20	3.0	51	6 072	6.053	0.019	6.0618	51	4.704	4.698	0.006	4.6995	1.3623	0.0007794	3.040	108.90	0.0007757	109 16
" 12th, A.M..	(X)	10.0	11.40	49	5.903	5.900	0.003	5.9017	51	5.425	5.408	0.017	5.4193	0.4824	0.0002760	1.810	108.98	0.0002723	109.72
" 12th, P.M..	(XI)	1.20	3.0	51	5.892	5.884	0.008	5.8880	51	5.425	5.405	0.020	5.4129	0.4751	0.0002718	1.793	108.76	0.0002681	109.51
" 13th, A.M..	(XII)	10.0	11.40	51	6.402	6.399	0.003	6.4016	51	6.285	6.277	0.008	6.2803	0.1213	0.0000694	0.905	108.61	0.0000657	111.64
" 13th, P.M..	(XIII)	1.20	3.0	51	6.398	6.395	0.003	6.3968	51	6 281	6.275	0.006	6.2776	0.1192	0.0000682	0.901	109.05	0.0000645	112.15
" 14th, A.M..	(XIV)	10.0	11.40	51	6.163	6.156	0.007	6.1591	51	6.106	6.098	0.008	6.1013	0.0578	0.0000331	0.605	105 27	0.0000294	111.74
" 14th, P.M..	(XV)	1.20	3.0	51	6.153	6.150	0.003	6.1523	51	6.097	6.094	0.003	6.0952	0.0571	0.0000327	0.595	104.12	0.0000290	110.61
Oct. 4th, A.M..	(XVI)	10.0	11.40	51	6.688	6.625	0.063	6.6579	51	4 852	4.828	0.024	4.8434	1.8145	0.0010381	3.472	107.76	0.0010344	107.95
" 4th, P.M..	(XVII)	1.20	3.0	51	6.770	6.728	0.042	6.7527	51	4.874	4.857	0.017	4.8654	1.8873	0.0010797	3.533	107.53	0.0010760	107.72
" 5th, A.M..	(XVIII)	10.0	11.40	51	5.422	5.395	0.027	5.4073	51	5.196	5.177	0.019	5.1851	0.2222	0.0001271	1.228	108.88	0.0001234	110.51

Oct. 5th, P.M..	(XIX)	1.20	3.0	51	5.383	5.375	0.008	5.3793	51	5.177	5.162	0.015	5.1684	0.2109	0.0001207	1.192	108.55	0.0001170	110.26
" 6th, A.M..	(XX)	10.0	11.40	51	6.024	6.020	0.004	6.0221	51	6.002	5.997	0.005	6.0601	0.0220	0.0000126	0.332	93.59	0.0000089	111.50
" 6th, P.M..	(XXI)	1.20	3.0	51	6.018	6.011	0.007	6.0141	51	6.012	5.992	0.020	5.9972	0.0169	0.0000097	0.327	105.24	0.0000060	134.15
NORTH PIPE TUBERCULATED.—LENGTH, 1 748.10 FT.																			
" 18th, A.M..	(XXII)	10.0	11.40	51	6.755	6.731	0.024	6.7418	51	4.895	4.854	0.041	4.8804	1.8614	0.0010648	3.412	104.57	0.0010600	104.80
" 18th, P.M..	(XXIII)	1.20	3.0	50	6.913	6.843	0.070	6.8776	51	4.945	4.899	0.046	4.9179	1.9597	0.0011210	3.510	104.83	0.0011162	105.05
" 19th, A.M..	(XXIV)	10.0	11.40	51	5.493	5.469	0.024	5.4795	51	4.530	4.505	0.025	4.5200	0.9595	0.0005489	2.470	105.41	0.0005441	105.87
" 19th, P.M..	(XXV)	1.20	3.0	51	5.461	5.443	0.018	5.4510	51	4.515	4.499	0.016	4.5059	0.9451	0.0005406	2.440	104.92	0.0005358	105.38
" 20th, A.M..	(XXVI)	10.0	11.40	51	5.959	5.919	0.010	5.9528	51	5.923	5.913	0.010	5.9177	0.0351	0.0000201	0.416	92.84	0.0000153	106.25
" 20th, P.M..	(XXVII)	1.20	3.0	51	5.943	5.939	0.004	5.9410	51	5.910	5.909	0.001	5.9092	0.0318	0.0000182	0.402	94.27	0.0000134	109.66
" 30th, A.M..	(XXVIII)	10.0	11.40	51	5.348	5.343	0.005	5.3447	50	5.149	5.133	0.016	5.1405	0.2042	0.0001168	1.135	105.06	0.0001120	107.26
" 30th, P.M..	(XXIX)	1.20	3.0	51	5.353	5.351	0.002	5.3519	51	5.152	5.135	0.017	5.1446	0.2073	0.0001186	1.138	104.50	0.0001138	106.66
NORTH PIPE CLEAN.—LENGTH, 1 748.10 FT.																			
Nov. 30th, A.M..	(XXX)	10.0	11.40	46	6.189	6.175	0.014	6.1797	51	5.001	4.981	0.020	4.9906	1.1891	0.0006802	3.674	140.87	0.0006754	141.36
" 30th, P.M..	(XXXI)	1.20	3.0	46	6.186	6.183	0.003	6.1842	51	5.002	4.983	0.019	4.9920	1.1922	0.0006820	3.683	141.02	0.0006772	141.51
Dec. 1st, A.M..	(XXXII)	10.0	11.40	46	5.095	5.089	0.006	5.0915	36	4.531	4.514	0.017	4.5239	0.5676	0.0003247	2.472	137.20	0.0003199	138.21
" 1st, P.M..	(XXXIII)	1.20	3.0	47	5.090	5.083	0.007	5.0858	51	4.533	4.520	0.013	4.5257	0.5601	0.0003204	2.476	138.33	0.0003156	139.37
" 7th, A.M..	(XXXIV)	10.0	11.40	46	6.538	6.525	0.013	6.5312	51	6.408	6.391	0.017	6.4004	0.1308	0.0000748	1.113	128.69	0.0000700	132.98
" 7th, P.M..	(XXXV)	1.20	3.0	46	6.549	6.536	0.013	6.5442	51	6.414	6.401	0.013	6.4086	0.1356	0.0000776	1.128	128.10	0.0000728	132.21
" 8th, A.M..	(XXXVI)	10.0	11.40	46	6.018	6.010	0.008	6.0133	51	5.995	5.989	0.006	5.9917	0.0216	0.0000124	0.395	112.46	0.0000076	143.32
" 8th, P.M..	(XXXVII)	1.20	3.0	49	6.024	6.018	0.006	6.0220	51	5.997	5.992	0.005	5.9941	0.0279	0.0000160	0.398	99.56	0.0000112	118.78
" 13th, A.M..	(XXXVIII)	10.0	11.40	85	5.905	5.878	0.027	5.8929	51	4.894	4.865	0.029	4.8783	1.0146	0.0005804	3.386	140.55	0.0005756	141.13
" 13th, P.M..	(XXXIX)	1.20	3.0	97	5.908	5.873	0.035	5.8912	51	4.837	4.862	0.025	4.8764	1.0148	0.0005805	3.387	140.58	0.0005757	141.16
" 14th, A.M..	(XL)	10.0	11.40	48	6.188	6.178	0.010	6.1823	51	6.132	6.124	0.008	6.1281	0.0542	0.0000310	0.659	118.43	0.0000262	128.69
" 14th, P.M..	(XLI)	1.20	3.0	48	6.186	6.179	0.007	6.1819	50	6.129	6.117	0.012	6.1232	0.0587	0.0000336	0.656	113.25	0.0000288	122.22
" 15th, A.M..	(XLII)	10.0	11.40	47	6.097	6.089	0.008	6.0925	51	6.064	6.052	0.012	6.0587	0.0338	0.0000193	0.535	121.74	0.0000145	140.16
" 15th, P.M..	(XLIII)	1.20	3.0	47	6.102	6.096	0.006	6.0982	51	6.061	6.054	0.007	6.0568	0.0414	0.0000237	0.539	110.80	0.0000189	123.92
" 24th, A.M..	(XLIV)	10.0	11.40	93	6.211	6.175	0.036	6.1943	46	2.992	2.952	0.040	2.9638	3.2305	0.0018480	6.141	142.84	0.0018432	143.02
" 24th, P.M..	(XLV)	1.20	3.0	49	6.183	6.163	0.020	6.1733	46	3.000	2.953	0.047	2.9692	3.2041	0.0018329	6.123	143.03	0.0018281	143.22
1895.																			
Jan. 10th, A.M..	(XLVI)	10.0	11.40	46	6.030	6.021	0.009	6.0261	46	4.832	4.720	0.112	4.7773	1.2491	0.0007145	3.723	139.28	0.0007097	139.75
" 10th, P.M..	(XLVII)	1.20	3.0	46	6.051	6.033	0.018	6.0426	46	4.812	4.707	0.105	4.7654	1.2772	0.0007306	3.723	137.75	0.0007258	138.20
" 11th, A.M..	(XLVIII)	10.0	11.40	46	7.345	7.311	0.034	7.3282	46	5.193	5.112	0.081	5.1513	2.1769	0.0012453	4.973	140.92	0.0012405	141.19
" 16th, A.M..	(XLIX)	10.0	11.40	46	7.963	7.863	0.100	7.9107	46	3.419	3.387	0.032	3.4056	4.5051	0.0025771	7.202	141.86	0.0025723	141.99
" 16th, P.M..	(L)	1.20	3.0	46	8.061	7.984	0.077	8.0204	47	3.512	3.358	0.154	3.4360	4.5844	0.0026225	7.245	141.47	0.0026177	141.60
SOUTH PIPE TUBERCULATED.—LENGTH, 1 747.96 FT.																			
Jan. 23d, A.M..	(LI)	10.0	11.40	51	6.135	6.113	0.022	6.1249	51	4.180	4.122	0.058	4.1536	1.9713	0.0011278	3.679	109.55	0.0011241	109.73
" 23d, P.M..	(LII)	1.20	3.0	51	6.163	6.144	0.019	6.1524	51	4.232	4.132	0.100	4.1735	1.9789	0.0011321	3.704	110.09	0.0011284	110.27
" 24th, A.M..	(LIII)	10.0	11.40	51	6.869	6.835	0.034	6.8538	51	3.271	3.232	0.039	3.2542	3.5996	0.0020593	4.980	109.74	0.0020556	109.84
" 24th, P.M..	(LIV)	1.20	3.0	51	6.935	6.916	0.019	6.9275	51	3.360	3.202	0.158	3.2765	3.6510	0.0020887	5.003	109.47	0.0020850	109.57
" 25th, A.M..	(LV)	10.0	11.40	51	7.150	7.128	0.022	7.1362	51	2.721	2.565	0.156	2.6047	4.5315	0.0025925	5.578	109.56	0.0025888	109.64
" 25th, P.M..	(LVI)	1.20	3.0	51	7.204	7.188	0.016	7.1961	51	2.623	2.601	0.022	2.6106	4.5855	0.0026233	5.591	109.15	0.0026196	109.23

TABLE No. 5.—Comparison Between Siphon and Terminal Chamber Weirs.

Using Bazin's values of m in the formula $Q = m\,L\,H\sqrt{2\,g\,H}$; $g = 32.17$.

Observations at Siphon Weir.

1	2	3	4		5	6	7	8	9	10
Date.	Number of experiment.	Temperature of water. Fahrenheit.	Time of beginning.	Time of ending.	Number of observations.	Mean height of water on weir. H. Feet.	Mean length of weir. L. Feet.	Coefficient from table. m.	Quantity in cubic feet per second. Q.	Ratio of quantity at siphon weir to quantity at terminal weirs.
1894.										
November 13th	1	38.8°	9.00 A.M.	12.20 P.M.	41	1.8897	4.9693	0.4429	45.860	0.9915
" 15th	2	38.7°	9.00 A.M.	1.40 P.M.	57	1.8748	4.9693	0.4427	45.298	0.9926
" 16th	3	39.0°	9.00 A.M.	1.40 P.M.	57	1.8790	4.9693	0.4427	45.450	0.9848
" 30th	4	35.5°	10.00 A.M.	11.40 A.M.	51	1.8979	4.9693	0.4430	46.169	0.9919
December 1st	5	35.5°	10.00 A.M.	11.40 A.M.	51	1.4699	4.9700	0.4373	31.068	0.9884
" 13th	6	....	10.00 A.M.	11.20 A.M.	41	1.8021	4.9694	0.4417	42.593	0.9924
" 21st	7	....	9.00 A.M.	12.20 P.M.	81	1.9392	4.9692	0.4435	47.737	0.9783
	Mean									0.9886

Observations at Terminal Weirs.

1	2	3	4		5	6	7	8	9	10	11	12	13	14	15	16
					East Weir.					West Weir.						
Date.	Number of experiment.	Temperature of water. Fahrenheit.	Time of beginning.	Time of ending.	Number of observations.	Mean height of water on weir. *H.* Feet.	Mean length of weir. *L.* Feet.	Coefficient from table. *m.*	Quantity in cubic feet per second. *Q.*	Number of observations.	Mean height of water on weir. *H.* Feet.	Mean length of weir. *L.* Feet.	Coefficient from table. *m.*	Quantity in cubic feet per second. *Q.*	Total quantity in cubic feet per second.	Total quantity less 0.558 cu. ft. per second leakage. Cubic feet per second.
1894.			P M	P.M.												
November 13th.....	1	....	1.40	4.58	50	1.0720	6.3259	0.4284	24.127	50	1.0848	5.8411	0.4285	22.684	46.811	46.253
" 15th.....	2	39.1°	2.20	7.00	141	1.6403	6.3238	0.4335	46.195	....	.....	.....	.....	.....	46.195	45.637
" 16th.....	3	39 1°	2.20	7.00	....	.....	.....	.....	.....	140	1.7399	5.8396	0.4345	46.709	46.709	46.151
" 30th.....	4	36.1°	3.20	5.00	51	1.6613	6.3237	0.4337	47.106	....	.....	.....	.....	.....	47.106	46.548
December 1st	5	36.1°	4.30	6.10	....	.....	.....	.....	.....	51	1.3594	5.8408	0.4308	31.990	31.990	31.432
" 13th.....	6	36.6°	3.00	4.20	40	1.0206	6.3261	0.4280	22.393	41	1.0339	5.8411	0.4281	21.086	43.479	42.921
" 21st.....	7	37.1°	1.40	5.00	100	1.1089	6 3258	0.4287	25.401	101	1.1244	5.8410	0.4288	23.953	49.354	48.796

TABLE No. 6.—Water Level Observations.

Piezometer Gauges.

1	2	3		4	5	6	7	8	
Date.	Number of experiment.	Time of beginning.	Time of ending.	West Piezometer.		East Piezometer.		Differences. E. piezometer reading minus, W. piezometer reading feet.	
				Number of observations.	Mean reading. Feet.	Number of observations.	Mean reading. Feet.		
				North Pipe.					
1894.									
September 7th, P. M.	1	1.20	3.00	46	4.3046	45	4.2909	−0.0137	
" 11th, P. M.	2	1.20	3.00	50	5.3540	50	5.3421	−0.0119	
" 12th, A. M.	3	10.00	11.40	50	4.2514	50	4.2528	0.0014	
" 12th, P. M.	4	1.20	3.00	50	5.4396	50	5.4335	−0.0061	
" 13th, A. M.	5	10.00	11.40	50	3.9089	50	3.9105	0.0016	
" 13th, P. M.	6	1.20	3.00	50	3.9096	50	3.9060	−0.0036	
" 14th, A. M.	7	10.00	11.40	50	3.8730	50	3.8742	0.0012	
" 14th, P. M.	8	1.20	3.00	50	3.8747	50	3.8735	−0.0012	
October 4th, A. M.	9	10.00	11.40	50	3.9531	50	3.9528	−0.0003	
" 4th, P. M.	10	1.20	3.00	50	4.0440	50	4.0408	−0.0032	
" 5th, A. M.	11	10.00	11.40	50	4.2414	50	4.2368	−0.0046	
" 5th, P. M.	12	1.20	3.00	50	4.2392	50	4.2345	−0.0047	
" 6th, A. M.	13	10.00	11.40	50	4.2508	50	4.2559	0.0051	
" 6th, P. M.	14	1.20	3.00	50	4.2481	50	4.2503	0.0022	
	Mean							−0.0027	±0.00097
	Vernier correction							+0.0008	
	Correction for zero of scale							= −0.0019	

SOUTH PIPE.

1894.										
October	18th, P. M.	15	1.20	3.00	50*	.....	50*	.....	.....	
"	19th, A. M.	16	10.00	11.40	208	5.0639	195	5.0658	0.0019	
"	19th, P. M.	17	1.20	3.00	200	5.0425	200	5.0428	0.0003	
"	20th, A. M.	18	10.00	11.40	187	4.8910	194	4.8917	0.0007	
"	20th, P. M.	19	1.20	3.00	177	4.8637	204	4.8610	—0.0027	
"	30th, A. M.	20	10.00	11.40	201	3.7612	201	3.7608	—0.0004	
"	30th, P. M.	21	1 20	3.00	201	3.7598	201	3.7596	—0.0002	
November	30th, A. M.	22	10.00	11.40	50*	.....	50*	.....	.....	
"	30th, P. M.	23	1.20	3.00	158	4.2604	201	4.2633	0.0029	
December	1st, A. M.	24	10.00	11.40	188	4.0946	199	4.0908	—0.0038	
"	1st, P. M.	25	1.20	3.00	197	4.0921	197	4.0899	—0.0022	
"	7th, A. M.	26	10.00	11.40	201	5.9003	201	5.9000	—0.0004	
"	7th, P. M.	27	1.20	3.00	201	5.7739	201	5.7704	—0.0035	
"	8th, A. M.	28	10.00	11.40	201	5.4098	201	5.4052	—0.0046	
"	8th, P. M.	29	1.20	3.00	197	5.3845	201	5.3818	—0.0027	
"	13th, A. M.	30	10.00	11.40	201	6 4096	201	6.4067	—0.0029	
"	13th, P. M.	31	1.20	3.00	201	6.3653	201	6.3638	—0.0015	
"	14th, A M.	32	10.00	11.40	201	5.8852	191	5.8831	—0.0021	
"	14th, P. M	33	1.20	3.00	198	5.8622	201	5 8601	—0.0021	
"	15th, A. M.	34	10.00	11.40	201	5.3347	198	5.3302	—0.0045	
"	15th, P. M.	35	1.20	3.00	197	5.3223	198	5.3218	—0.0005	

Mean	—0.0015	±0.00032
Vernier correction	—0.0005	
Correction for zero of scale	—0.0020	

* Observations not used in obtaining mean, because they were taken at two-minute intervals instead of 30 second intervals, and are therefore of less weight than the others.

TABLE No. 7.—Water Level Observations.

Tube Piezometer Gauges.

1	2	3		4	5	6	7	8	
				West Tube Piezometer.		East Tube Piezometer.			
Date.	Number of experiment.	Time of beginning.	Time of ending.	Number of observations.	Mean reading. Feet.	Number of observations.	Mean reading. Feet.	Differences. E. piezometer reading minus. W. piezometer reading feet.	
				North Pipe.					
1894.									
September 7th, P.M.	1	1.20	3.00	50	4.2924	50	4.2865	—0 0059	
" 11th, P.M.	2	1.20	3.00	50	5.3512	50	5.3389	—0.0123	
" 12th, A.M.	3	10.00	11.40	49	4 2626	50	4.2536	—0.0090	
" 12th, P M.	4	1.20	3.00	50	5.4413	50	5.4295	—0 0118	
" 13th, A.M.	5	10.00	11.40	50	3.9197	50	3.9080	—0.0117	
" 13th, P.M.	6	1.20	3.00	50	3.9119	50	3.9010	—0.0109	
" 14th, A.M.	7	10.00	11.40	49	3.8847	50	3.8761	—0.0086	
" 14th, P.M.	8	1.20	3.00	47	3.8834	50	3.8756	—0 0078	
October 4th, A.M.	9	10.00	11.40	50	3.9557	50	3.9466	—0.0091	
" 4th, P M.	10	1.20	3.00	50	4.0442	49	4.0376	—0.0066	
" 5th, A.M.	11	10.00	11.40	50	4.2382	50	4.2328	—0.0054	
" 5th, P.M.	12	1.20	3 00	50	4.2333	50	4.2253	—0.0080	
" 6th, A.M.	13	10.00	11.40	50	4.2590	50	4.2517	—0.0073	
" 6th, P.M.	14	1 20	3.00	50	4.2548	50	4.2534	—0.0014	
	Mean	.....	.....	.....	.....	.....	.....	—0.0083	±0.00053

SOUTH PIPE.

1894.									
October 18th, P.M.	15	1.20	3.00	51	5.2234	50	5·2128	—0.0106	
" 19th, A.M.	16	10.00	11.40	50	5.0639	50	5.0585	—0.0054	
" 19th, P.M.	17	1.20	3.00	50	5.0375	50	5.0312	—0.0063	
" 20th, A.M.	18	10.00	11.40	50	4.8869	50	4.8796	—0.0073	
" 20th, P.M	19	1.20	3.00	50	4.8516	50	4.8452	—0.0064	
" 30th, A.M.	20	10.00	11.40	50	3.7623	50	3.7592	—0.0031	
" 30th, P.M.	21	1.20	3.00	50	3.7633	50	3.7569	—0.0064	
November 30th, A.M.	22	10.00	11.40	45	4.3253	50	4.3151	—0.0102	
" 30th, P.M.	23	1.20	3.00	45	4.2652	50	4.2555	—0.0097	
December 1st, A.M.	24	10.00	11.40	45	4.0880	50	4.0852	—0.0028	
" 1st, P.M.	25	1.20	3.00	45	4.0848	50	4.0826	—0.0022	
" 7th, A.M.	26	10.00	11.40	45	5.9025	50	5.8938	—0.0087	
" 7th, P.M.	27	1.20	3.00	45	5.7721	50	5.7650	—0.0071	
" 8th, A.M.	28	10.00	11.40	45	5.4064	50	5.4011	—0.0053	
" 8th, P.M.	29	1.20	3.00	45	5.3861	50	5.3765	—0.0096	
" 13th, A.M.	30	10.00	11.40	90	6.4111	50	6.4082	—0.0029	
" 13th, P.M.	31	1.20	3.00	95	6 3764	50	6.3627	—0.0137	
" 14th, A.M.	32	10.00	11.40	47	5.8824	50	5.8789	—0.0035	
" 14th, P.M.	33	1.20	3.00	47	5.8632	49	5.8543	—0.0089	
" 15th, A.M.	34	10.00	11.40	50	5.3299	50	5.3296	—0.0003	
" 15th, P.M.	35	1.20	3.00	46	5.3214	50	5.3148	—0.0066	
	Mean	.....	.....	.....	.....	.....	.....	—0.0065	±0.00049

DISCUSSION.

Mr. Gould. E. Sherman Gould, M. Am. Soc. C. E.—Such elaborate and painstaking experiments as those conducted and described by the author are of the highest scientific value. In the present case, however, one circumstance seems to impair the usefulness of the results, namely, that the line experimented on, being only about 420 diameters in length, cannot be classed fairly as a long pipe. It is to be regretted, also, that the difference of level of the water surfaces in the two gatehouses was not recorded as well as that of the piezometric heights. This would have been a valuable addition to the data.

These experiments show unusually large volumes of water passing through the cleaned pipes at the higher velocities. The most natural explanation of this lies in the fact already stated, that the pipe is not really a long pipe, and that, therefore, with a very clean surface and relatively steep hydraulic gradient, the interior resistances did not have a chance to exert their full influence. It is to be feared that dangerous generalizations may arise from these experiments, and the formulas derived from them may be used incautiously in cases to which they do not apply.

As regards the choice of a safe, practical formula for the flow of water through long pipes, it may be laid down as a fundamental principle, that it should conform to the ordinary requirements of all practical engineering formulas.

The only test of the merit of an engineering formula is whether, in a vast majority of practical cases, it gives results below rather than above those commonly realized. If it does this, it is a good formula, no matter how it was derived, whether by elaborate experiment or mere guess. Moreover, a safe engineering formula cannot always be derived even from experiment, because the experiment is necessarily conducted under circumstances which do not or may not exist in practice. An elaborate experiment is useful to establish a law of nature, such as universal attraction or the mechanical equivalent of heat, but it cannot establish a practical engineering formula. That must be founded upon the general behavior of the working apparatus as commonly constructed. In this case it is the apparatus about which information is wanted and not the law.

As a safe, simple and practical formula for the flow of water through long pipes, it is believed that no other is so good as that of Darcy, and in making this assertion, the perfection of the experiments upon which it is founded is waived, and the statement based entirely upon the concordance of its results with those realized in practice. This formula is thus expressed:

$$V = \sqrt{D\,H \div C\,L} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

Whence,

$$Q = A\,\sqrt{D\,H \div C\,L} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

In these formulas, V is the velocity in feet per second, D is the diameter of the pipe in feet, H is the total head, L is the length of pipe, C is a numerical coefficient, Q is the discharge in cubic feet per second, and A is the area of pipe in square feet. Mr. Gould.

The coefficient C varies for clean pipes. from 0.00033 for a pipe 1 ft. in diameter to 0.00031 for one of 4 ft. For rough pipes the above values become 0.00066 and 0.00062 respectively.

Formula (2) may be written thus:

$$Q = \sqrt{0.617\ D^5\ H \div C\ L}$$

If C in this formula is assumed to be 0.0003085 for smooth and 0.000617 for rough pipes, and if L is taken as 1 000, and h as the fall or head per 1 000, there results for smooth pipes:

$$Q = \sqrt{2\ h\ D^5} \quad \ldots\ldots\ldots\ldots \quad (3)$$

$$D = \sqrt[5]{Q^2 \div 2\ h} \quad \ldots\ldots\ldots\ldots \quad (4)$$

and

$$Q = \sqrt{D^5\ h} \quad \ldots\ldots\ldots\ldots \quad (5)$$

$$D = \sqrt[5]{Q^2 \div h} \quad \ldots\ldots\ldots\ldots \quad (6)$$

for rough pipes. Also

$$V = \sqrt{3.24\ D\ h} \quad \ldots\ldots\ldots\ldots \quad (7)$$

for smooth pipes, and

$$V = \sqrt{1.62\ D\ h} \quad \ldots\ldots\ldots\ldots \quad (8)$$

for rough pipes.

It will be seen that the extreme variation between a smooth and a rough pipe is here assumed as $\sqrt{2}$, or, say, 40% greater in the former than in the latter. Or to state it in another manner, the discharge of the rough pipe is 0.707, say 70%, of that of the smooth pipe.

To submit these formulas to the only true test, that of practical results, they may be compared with the examples given by Edmund B. Weston, M. Am. Soc. C. E., in his valuable paper, "The Results of Investigations Relative to Formulas for the Flow of Water in Pipes." *

The examples taken are for diameters of 12, 16, 20, 30, 36 and 48 ins. The author's observed results have been added for comparison.

In the table on page 278, the first column gives the name of the experimeter; the second, the nature of the pipe; the third, the diameter in inches; the fourth, the fall per 1 000; the fifth, the observed velocity; and the sixth and seventh, the velocities according to Darcy's simplified formulas for smooth pipes (7) and rough pipes (8), respectively. These two formulas are interchangeable by the use of the coefficient 1.41 or 0.707, applied respectively to the formula for rough or smooth pipes.

It will be perceived that in all the range of practical examples taken from Mr. Weston's paper, the observed velocities lie between the extremes represented by formulas (7) and (8).

* See *Transactions*, Vol. xxii, p. 1.

Mr. Gould.

Authority.	Character of pipe.	Diameter, inches.	Fall per 1 000.	Observed velocity, feet per second.	$V = \sqrt{3.24\,Dh}$	$V = \sqrt{1.62\,Dh}$
Simpson	Cast iron, less than seven years' service	12	0.77	1.45	1.58	1.12
"	Cast iron, less than seven years' service	12	7.70	4.35	5.00	3.53
"	Cast iron, less than four years' service	12	6.97	4.11	4.75	3.36
Bonn Water-Works	New cast iron, asphalted	12+	1.20	1.56	1.97	1.39
"	"	12	3.66	3.10	3.44	2.43
Edinburgh Water-Works	Cast iron, eight or nine years' service	16	8.92	5.25	6.20	4.38
"	Cast iron, eight or nine years' service	16	48.23	14.51	14.40	10.18
Brush	Cast iron, tarred, five years' service	20	0.729	2.00	1.98	1.40
"	Cast iron, tarred, five years' service	20	1.49	2.76	2.83	2.00
"	Cast iron, tarred, five years' service	20	1.797	3.00	3.10	2.19
Simpson	Cast iron, two or three years' service	30	0.462	1.77	1.93	1.36
Greene	Cast iron, heavily tuberculated	36	1.80	3	4.16	2.95
Fitzgerald	South pipe, tuberculated	48	1.02	3.486	3.64	2.57
"	"	48	2.62	5.591	5.83	4.12
"	North pipe, tuberculated	48	0.0175	0.402	0.597	0.422
"	"	48	1.136	3.51	3.84	2.71
"	North pipe, clean	48	1.83	6.141	4.87	3.44
"	"	48	2.62	7.245	5.83	4.12
"	"	48	0.5765	3.386	2.73	1.93
"	"	48	0.0217	0.535	0.531	0.375
"	"	48	0.0123	0.398	0.399	0.282
					$V = R\sqrt{\frac{100\,h}{6.6\,R + 0.46}}$	
Clark	Brick tunnel	90	0.501	3.77	3.70	
"	"	90	0.582	3.93	3.99	

On the other hand, a marked difference is observable in the case of the author's cleaned pipes at the higher velocities, which greatly exceed the velocities given by formula (7), with which, however, they nearly agree for very low velocities of a few inches per second. The velocities for the tuberculated pipes fall between those given by formulas (7) and (8), as might be expected.

Setting aside the very low velocities, due to falls of less than 2 ins. per mile, which are of little practical interest, the fact remains that out of a large body of actual measurements the author's smooth pipes alone give results differing widely, in excess, from those given by Darcy's formula for similar pipes. Assuming the data in all cases to be correct, it seems that the most reasonable explanation is that already

suggested, namely, that the line was not sufficiently long for the full development of the resistances of the interior surface. The only other rational explanation would be an abnormal degree of smoothness of the surface, and either circumstance would annul the value of the experiments as practical guides. Mr. Gould.

In the case of the 90-in. brick tunnel, the observed velocity agrees admirably with that given by Darcy's formula for such conduits. This formula is conveniently rendered—

$$V = R\sqrt{\frac{100\,h}{6.6\,R + 0.46}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (9)$$

in which R is the mean hydraulic radius.

This last formula (9), which seems to be justified by other observations also, gives higher velocities than (7). Indeed, it seems reasonable that a brick-lined conduit, which is always laid to a true descending grade, with few bends and with a uniform interior surface, should give a higher rate of velocity than an iron pipe line, as generally laid.

The preceding table includes no diameters below 12 ins., because from and even at that diameter the coefficient C begins to increase so rapidly that the simplified formula would give too high comparative velocities. For smaller diameters formula (1) should be used, with the proper value for C, as established by Darcy. Mr. Weston's observations on a 6-in. pipe, given in his paper already referred to, justify the use of the simplified formula for pipes of this diameter also. In the absence of further observations, however, it would be best to use it for small diameters as an approximation only.

In view of the above facts, it would appear that Darcy's formula for rough pipes in its simplified form, given in equation (6), furnishes a safe rule to calculate diameters of cast-iron pipes from 12 to 48 ins., and that no formula giving smaller diameters should be trusted for a permanent water supply. It fulfils the essential condition of all working engineering formulas, that of containing a reasonable, but not extravagant, factor of safety.

CORRESPONDENCE.

Mr. Hering. RUDOLPH HERING, M. Am. Soc. C. E.—No contribution of results obtained by actual experiment in the domain of hydraulic engineering can be more welcome than one referring to the discharging capacity of pipes under different conditions, on account of the money value which is often represented by them. There still seems to be a lack of full appreciation of the inductions that have been drawn from the few but well-authenticated gaugings which have been made, and therefore more evidence is needed to increase confidence in them. The author has rendered a good service to the profession in publishing a detailed account of his late experiments. They were evidently made with much care, and the results obtained are correspondingly accurate and valuable.

It is the writer's purpose to say a few words regarding the latter part of the paper, which is devoted to an expression of the results by formulas. Such algebraic aids are often of great value to a practical engineer, but sometimes they are the reverse, and, unless thoroughly understood, are liable, under the cloak of apparent exactness, to misrepresent facts and cause failures.

The author has made the simple Chezy formula the basis of his study, to which the writer desires to give a strong endorsement, not only because of its simplicity, but also because of its rational construction and practical usefulness. Unfortunately the less simple Weisbach and other "$2\,g\,h$" formulas are still occasionally used with apparent preference, probably because their theoretical origin is indicated a little more clearly to the eye, but to the practical man they are often unnecessarily clumsy and perhaps occasionally misleading.

The Chezy, or more properly the Brahms, formula is: Mean velocity $= c\sqrt{r\,s}$, in which r is the mean radius, s is the sine of slope, and the coefficient c, as originally supposed, was a constant, and was ascertained by experiment. For rough results, amply sufficient for many cases, this expression has been used for over a century, Brahms having first suggested it in 1753. With the development of hydraulics and the demand for greater precision, the so-called coefficient c was found to require modification when applied under different conditions. In other words, it was found by more extensive experience not to be a constant, but a variable, quantity.

Through the classic series of experiments conducted by Darcy and Bazin in France, it was found that the value of the coefficient varied greatly with the character or roughness of the wetted perimeter and also somewhat with the mean radius of the section. Humphreys and Abbot, in their experiments on the flow of the Mississippi River, found that the coefficient c varied with the hydraulic slope. Mr. Ganguillet,

city engineer of Berne, Switzerland, and a capable mathematician, suggested an algebraic expression for the coefficient c which would embody these three variations, and, together with Mr. Kutter, his assistant, developed what is now known as the Kutter formula. This formula, if applied to the experiments of Darcy and Bazin, is found not only to represent them well, but, on the average, actually better than Darcy's own formula. For ascertaining the flow of water in channels of regular section a new and more accurate expression was therefore given for the original coefficient c. Its variation with the mean radius and with the slope was given a mathematical form, because these quantities could be suitably expressed, and a new constant, the coefficient n, was introduced, which was intended to represent a certain definite degree of roughness of the wetted perimeter. Mr. Hering.

A much greater refinement was thus obtained, and the formula could be applied with greater confidence as regards the result. At first the authors of the Kutter formula divided all possible cases into classes or categories (as did Darcy and Bazin), and suggested six different values for the coefficient of roughness n, beginning with smooth cement or planed boards and ending with streams, the beds of which were covered with detritus and aquatic plants. Later these six classes were given up. The advantages of this new formula were grasped quickly by the engineering profession, and it gradually supplanted the old formulas for general use.

In recent years, however, through the further development of engineering science, the demand for greater refinements, for greater economy in getting better results with less expenditure of money, has put the Kutter formula in a similar position to that occupied by the Chezy formula thirty years ago. The coefficient n, which was first considered to be a constant quantity, and which roughly can be properly considered as such, is also found to vary, though between much smaller limits than the original coefficient c.

To illustrate this statement by the simile of a decimal fraction, suppose the Chezy formula gave results that could safely be expressed by units only, the greater refinement of the Kutter formula gave results which could be safely expressed in tenths of a unit. At the present time there seems to be need of a formula which will give safe results in hundredths of a unit. Gaugings are being made with greater precision. It is more necessary to-day that water courses and pipe lines should give the greatest discharges with the least possible outlay of money. Works built in recent years on the assumption, continuing to use the above simile, that accuracy up to tenths of a unit was sufficient, have, in more than one case, disastrously affected invested capital.

An important question to-day is how further improvements can be made in more accurately forecasting the mean flow of water in projected channels. By some radically new formula? The writer thinks

Mr. Hering. not. It seems there are already too many, and thereby have caused some little confusion rather than elucidation on the subject. To the writer's mind, the proper course to pursue appears to be the following: Starting with the old and simple Chezy formula as a foundation, its coefficient c may be considered a constant quantity for rough approximations, perhaps distinguishing between pipes, artificial channels and rivers. For closer approximations the Kutter formula may be used, which expresses mathematically some of the variations of this coefficient, and introduces a new coefficient n, which is readily ascertained in practice, because it represents mainly the degree of roughness of perimeter and can be interpreted by gaugings made in similar channels. For still closer approximation it is necessary to consider the variations of this coefficient of roughness in one and the same channel, with its slope and perhaps also with its size and shape. Attention should be paid to these variations when safer and more precise results are desired, rather than to attempts to find a radically new formula.

Realizing such a necessity in the future, the writer endeavored about ten years ago to plot curves from actual gaugings, in order to ascertain the nature of the variation of this coefficient of roughness n. Sufficient information was obtained to indicate the *rationale* and practicability of such a course. To facilitate application in practice the writer compiled a very large collection of gaugings* from the case of pipes to that of the largest rivers, and arranged them with the special purpose of showing the variation of Kutter's coefficient n with the variation of the slope in one and the same channel. By giving some attention to this variation, as indicated by known and carefully made gaugings, an engineer will be enabled to forecast the discharge of a channel much more safely, and by the constantly increasing stock of gaugings, including cases of every characteristic kind of surface, he will be able to get closer and closer to the truth, and thus better fulfil the increasing demands for accuracy and safety on the part of invested capital.

It is quite possible that the accumulation of gaugings, as they enabled Ganguillet and Kutter to devise a formula for the variation of the Chezy constant, will, through the greater number of more accurate gaugings since made available, enable some one to devise a formula for the variation of the Kutter constant. Until then, the available gaugings themselves must be used and interpreted by the engineer in applying them to each new case.

It has recently been stated by a member of this Society that it would not be possible to determine reliable, variable coefficients of roughness to answer for all localities, because some iron pipes corrode much

* Embodied in the appendix to Ganguillet and Kutter's work on "The Flow of Water," translated by Rudolph Hering and John C. Trautwine, Jr.

faster than others and consequently the degree of roughness of the interior of an iron pipe in one locality, after a few years of service, might be quite different from what it would be in another locality, where for instance the water had a different chemical composition. Mr. Hering.

This statement is one of those which are apt to cause confusion of ideas. No formula can ever be devised nor can any rule be determined whereby the flow of water in pipes can be ascertained, when such a formula or rule must depend upon a gradual and complex change of conditions, such as subsequent corrosion, a subsequent deposit of scale or sediment, attachment of a slimy coating or a vegetable growth. The probable future extent of these contingencies must be determined by a mental and not by an algebraic process, and the skilful engineer will decide for what kind or degree of such contingencies he thinks it proper to provide in his special case, and select the coefficients in accordance with his decision. Judgment will be very materially aided if the laws according to which the coefficients are found to vary have been ascertained and recognized. Instead of considering only two or three so-called constants, each one applying to a general class of conduits, and each one embracing a great variety of conditions, the engineer will be able to draw safer conclusions if he has a continuously increasing scale of coefficients before him, along which scale he finds those coefficients which represent actual gaugings for definite conditions, and between which lies the coefficient that must apply to his new case. By means of his judgment, then, weighing the differences between the conditions of his new case and those for which actual gaugings exist, he can locate a value along that scale for the coefficient which applies most nearly to his new case.

It is unfortunate that the author gives also three entirely new formulas of the exponential class to fit his three series of experiments, without a special warning, which he perhaps thought was unnecessary, that they should not be applied to other cases. Such a possibility should not always be excluded, because cases are occasionally found where such formulas have been misapplied. The exponential class offers easy adaptations for any short series of experiments, and thereby gives a semblance of accuracy to the formula and apparently justifies confidence, but when applied to other cases it is sometimes liable to hit far off the mark. It is much safer for practical purposes to use the old simple Chezy instead of the exponential form, and, according to the degree of accuracy desired or obtainable, vary the coefficient c according to the Kutter formula, and for finer work to vary also the coefficient n, in accordance with the results of carefully made gaugings in similar channels, to suit the conditions of the particular case in hand.

The author's conclusion that piezometric gauges can be depended upon as certainly as other appliances of precision is gratifying to note because of the facilities they offer for making gaugings of this class.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E.—The author's experiments bring out clearly the well known diminution in capacity of a pipe line by reason of interior tuberculation due to 15 to 20 years' service in soft water, the reduction in the velocity coefficient in this case being 30% in 16 years. This seems to accord fairly well with the old standard formulas for the difference between the coefficients for clean and rusty pipes, which are in the ratio of eleven to eight respectively. The tables seem to show a double loss due to tuberculation, which is a new feature; that is to say, a loss due to tuberculation proper and that due simply to increased velocity of flow such as naturally accompanies the gradual increase in demand for water.

As the author states, the Kutter formula's coefficient of roughness is badly in need of another sub-coefficient. This is a most important fact for engineers to study, and brings out a fundamental defect in all the old standard formulas.

Nearly every pipe line becomes more or less foul by long-continued use, and a certain loss in capacity for discharge must necessarily follow. The loss due to tuberculation proper, as well as the loss due strictly to increase in velocity, both act prejudicially as regards future capacity of any pipe line, the only difference between any two cases being one of degree.

The chief water supplies at and near San Francisco are more or less impregnated with salts of lime, and may be classed as hard. Tuberculation is rare, and only where soft waters are in use. Pipes which have been in service for 10 to 25 years generally show a lime-scale on the inner surface, making it somewhat rougher than the new surface of asphalt; but no material change in the value of c has been noted as yet for the larger size pipes.

The formulas deduced by the author seem to fit his cases well, but whether they are suitable to apply to other cases remains to be seen. This has been the main trouble with the old formulas; they were reliable only within narrow limits near the experiments on which they were based. The results of the author's experiments should be applied to modifying the expression for n in Kutter's formula by introducing a new variable V, determined by Darcy's formula as a first approximation.

Mr. Hawks. A. McL. Hawks, Jun. Am. Soc. C. E.—The author shows in Plate VIII, Figs. 1 and 2, two radically different types of tuberculation in the pipes. As he does not state the relative positions of the two points, the writer wishes to ask for them. In recovering mains which have been replaced by larger ones he has noticed that those pipes laid in the lowest depressions were usually most affected by tubercles, and is desirous of learning if the same was true in this case.

In Plate VIII, in the immediate foreground, is shown an almost complete circle of tubercles. Is this a joint of the pipe encrusted in this

manner? The writer has also noticed that the most vulnerable places in the coating of recovered pipes seemed to be at the joints. This is probably due to the sharp edges of both the hub and spigot on the inside of the pipe being liable to be nicked in handling, and the coating on these sharp edges is very easily bruised or worn off. The slightest spot in the coating is sufficient to produce a large tubercle in a few years. In some pipes the effect of tuberculation at the joints was to diminish the clear diameter from ¼ to ½ in., not a matter of much moment to a 48-in. pipe, but a serious thing for a small service or lateral. To such an extent has this been found true, that the writer has drawn plans for joints with curves of ¼-in. radius to replace the sharp edges mentioned, and has advised, to insure perfect coating of the pipes at the joints, that the space easily reached from the end, about 3 ins. in small pipes, be retouched just before laying, with some of the asphalt varnishes which are rapid driers. Mr. Hawks.

The author does not state if he measured the area or height of the largest of these tubercles. This would be useful as showing to what extent the clear opening of the pipe might be reduced in 20 years, for conditions might easily arise whereby a ring of the largest tubercles would be formed outside of the easily protected end joints, and thereby the flow reduced, not only by the additional friction head, but by an actual reduction of cross-section. The shape of the tubercles illustrated differs materially from those observed by the writer. In the author's plates they look like barnacles with serrated tops. In pipes which have been in use from three to ten years they are smooth, rounded humps, much resembling a blister in the coating, and in that stage are very thin and easily ruptured and removed. The writer has contemplated the use of a rattler such as is used for cleaning pipes in the oil regions, for the removal of green tubercles, if the term may be used, and desires the author's opinion as to whether such means would be effective with such growths as he removed. In most cases the conduits leading to smaller cities and towns are less than 24 ins. in diameter inside, which is about the limiting size in which any successful manual labor can be performed, and with such pipes the author's methods of cleaning are not applicable.

Edmund B. Weston, M. Am. Soc. C. E.—The writer has been greatly interested in reading the paper on account of the valuable data which it contains, and the corroboration by some of these data of ideas that he expressed a number of years ago in a paper* entitled, "The Results of Investigations Relative to Formulas for the Flow of Water in Pipes." In this paper the writer states that he has come to the conclusion that two formulas constructed by an eminent French civil engineer, the late Henry Darcy, were very well adapted for pipes having interior sides similar to new cast-iron pipes. One of Mr. Weston.

* See *Transactions*, Vol. xxii, p. 1.

Mr. Weston. these formulas the writer considered applicable in cases when the velocity of flow was less than 0.33 ft. per second, and the other, which is ordinarily known as Darcy's formula, the writer was of the opinion was a safe general formula for cases when the velocity of flow is greater than 0.33 ft. per second.

For certain reasons mentioned in the paper referred to, the writer adopted the coefficient of friction, ζ, in the following formula as a basis of comparison:

$$h = \zeta \frac{l}{d} \frac{v^2}{2g}$$

In which h = the loss of head due to friction, in feet.
l = the length of the pipe, in feet.
d = the internal diameter of the pipe, in feet.
v = the velocity per second of the water flowing in the pipe, in feet.
ζ = the coefficient of friction of the water flowing against the interior sides of the pipe, $= \frac{2gh}{v^2} \frac{d}{l}$
$2g = 64.326$.

The writer, therefore, in the discussion of the present paper will use as a basis of comparison the coefficient of friction ζ. The author states that for the tuberculated pipes, the formula $v = 108 \sqrt{I}$ fits the experiments fairly well for all heads. The coefficient ζ computed from this formula is 0.0221, and the coefficient ζ computed from Darcy's formula for a pipe 48 ins. in diameter is $\zeta = 0.0198920 + \frac{0.00166573}{d} = 0.02031$. The coefficient ζ computed from experimental data relative to a coal-tar coated cast-iron pipe that had been in service eight years, obtained in Scotland a number of years ago by Mr. James M. Gale, and given in the writer's paper*, is 0.0204.

The writer formed his conclusions relative to Darcy's formula by comparing it with the results of 46 experiments made by 13 different experimenters in the United States, England, Scotland, France and Germany. In making the experiments 25 different pipes were used, ranging from about 3 to 90 ins. in diameter, which were new pipe, or had been in service from two to thirteen years. Some of these pipes were coated with coal-tar and others were not.

The writer's advocacy of Darcy's formula has principally been for the purpose of determining the size of pipes in the first place, and it might seem illogical, without an explanation, that he should have

* See *Transactions*, Vol. xxii, p. 22.

drawn his conclusions from comparisons of results derived from experiments made with pipes which had been in service as great a length of time as thirteen years. The number of experiments that had been made with entirely new pipe, however, was too limited to make use of them alone, and the writer considered that not only was it essential to provide a pipe which would furnish the required supply immediately after it was laid, but that the future capacity of the pipe, to a certain extent, should also be taken into consideration. The advisability of doing this is set forth by the author's experiments, and those made by Mr. Weston.

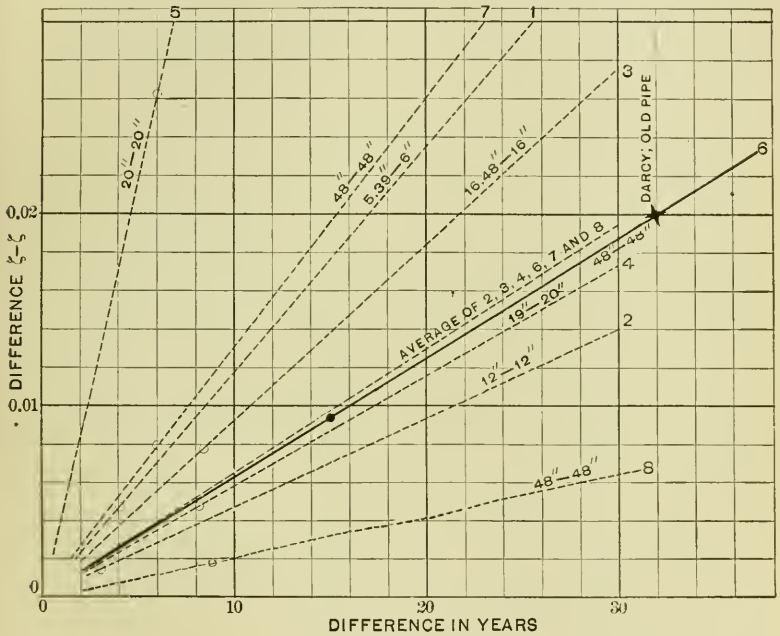


FIG. 24.

F. P. Stearns, M. Am. Soc. C. E.*, about two years after the same pipes were first laid, namely, the mean coefficient ζ computed from Mr. Stearns' experiments is 0.0126, the mean coefficient ζ computed from the author's experiments after the pipes were cleaned being 0.0128; and the mean coefficient ζ computed from his experiments immediately before the pipes were cleaned is 0.0221, showing an increase in the coefficient ζ of 0.0095 (0.0221 — 0.0126), or 75%, in fifteen years.

The increase in the coefficient ζ, which the writer has just mentioned, is shown by line No. 6, in Fig. 24.

* See *Transactions*, Vol. xiv, p. 1.

Mr. Weston. TABLE SHOWING THE INCREASE OF $\zeta - \zeta$ DUE TO YEARS OF SERVICE.

Diagram. Number.	Diameter. Inches.	Years of service.	Coefficient of friction. ζ.	Average. ζ.	Difference. $\zeta - \zeta$.	Average rate of increase of $\zeta - \zeta$ per 10 years.	Velocity. Feet per second.	Name of Experimenter and Kind of Pipe.
	5.39	.....	.0220	.....	.....	.....	2 50	Darcy.—New cast-iron pipe
			.0208	.....	.....	.....	4.19	
			.0204	.....	.....	.....	5.62	
		New.	.0203	.0207	.....	.....	6.88	
	6.00	.....	.0264	.....	.....	.....	4.70	Weston.—Cast-iron pipe coated by Smith's process.
			.0249	.....	.....	.....	7.25	
1...	.....	4	.0249	.0254	.0047	.0118	8.49	
	12.00	.....	.0276	.....	.....	.....	2.91	Simpson.—Cast-iron pipe.
			.0243	.....	.....	.....	2.91	
			.0259	.....	.....	.....	4.35	
		7	.0246	.0256	.....	.....	4.35	
	12.00	.....	.0263	.....	.....	.....	3.57	Simpson.—Cast-iron pipe.
2...	.....	4	.0277	.0270	.0014	.0047	4.11	
	16.48	.....	.0197	.....	.....	.....	2.48	Lampe.—New cast-iron pipe, coated with varnish.
			.0196	.....	.....	.....	2.71	
		New.	.0180	.0191	.....	.....	3.09	
	16.00	.....	.0277	.....	.....	.....	5.25	Edinburgh Water Company.—Cast-iron pipe.
3...	.....	8.5	.0261	.0269	.0078	.0092	6.82	
	19.00	.....	.0257	.....	.....	.....	2.06	Simpson.—Cast-iron pipe.
			.0244	.....	.....	.....	2.26	
			.0242	.....	.....	.....	2.52	
			.0248	.....	.....	.....	2.73	
		13	.0251	.0248	.....	.....	2.80	
	20.00	.....	.0197	.....	.....	.....	2.00	Brush.—Cast-iron pipe, coated with tar.
			.0188	.....	.....	.....	2.24	
			.0198	.....	.....	.....	2.36	
			.0200	.....	.....	.....	2.52	
			.0199	.....	.....	.....	2.68	
			.0210	.....	.....	.....	2.76	
			.0206	.....	.....	.....	2.92	
4...	.....	5	.0214	.0202	.0046	.0058	3.00	
	20.00	.....	.0568	.....	.....	.....	2.71	Darrach.—Cast iron pipe, coated with tar.
			.0539	.....	.....	.....	3.01	
			.0508	.....	.....	.....	3.31	
			.0482	.....	.....	.....	3.61	
			.0459	.....	.....	.....	3.91	
			.0437	.....	.....	.....	4.21	
			.0413	.....	.....	.....	4.51	
			.0394	.....	.....	.....	4.81	
5...	.....	11	.0376	.0464*	.0262	.0437	5.11	
	48.00	.....	.0120	.....	.....	.....	2.62	Stearns.—Cast-iron pipe, coated with coal-tar.
			0131	.....	.....	.....	3.74	
			.0128	.....	.....	.....	4.97	
		2	.0124	.0126	.....	.....	6.20	
6...	48.00	17	.....	.0221	.0095	.0063	.....	FitzGerald.—The same.
7...	48.00	8	.0204	.0204	.0078†	.0130	3.46	Gale.—Cast-iron pipe, coated with coal-tar.
8...	.....	.....	.....	.0204	.0017‡	.0019		

* Between data of Brush and Darrach.
† Between data of Stearns and Gale.
‡ Between data of FitzGerald and Gale.

In the above table, in addition to those mentioned, are given differences between coefficients ζ that have been computed from the

results of experiments made with pipes that had been in service different lengths of time in the United States, England, France and Germany. These differences are simply presented as a very slight verification of the law of increase shown by the experiments of Messrs. FitzGerald and Stearns, as the coefficients ζ for each diameter of pipe were worked out from experiments made at only one period with each pipe, the experiments not being repeated after the pipes had been in service a longer length of time; consequently, the differences are those of coefficients ζ computed from experiments made with different pipes, whereas those of Messrs. FitzGerald and Stearns were made with the same pipe. These differences are also plotted in Fig. 24 and have been grouped in order to show the increase in the coefficients ζ due to years of service, as follows: 5.39 and 6 ins.; 12 and 12 ins.; 16.48 and 16 ins.; 19 and 20 ins.; 20 and 20 ins.; 48 and 48 ins.; and 48 and 48 ins. The average of the six lines numbered 2, 3, 4, 6, 7 and 8, which the writer thinks are the most reliable, is also plotted in Fig. 24. Mr. Weston.

The coefficient ζ computed from Darcy's formula for old pipes is just double the coefficient ζ computed from Darcy's formula for new pipes (for a pipe 48 ins. in diameter, $2 \times 0.02031 = 0.04062$), and it is shown by plotting the difference between the coefficients ζ for old and new pipes ($0.04062 - 0.02031 = 0.02031$), upon line No. 6 of Fig. 24, that the increase 0.02031 corresponds to a length of time of about 32 years.

By the aid of these data the writer suggests a new method of applying Darcy's formula by which the number of years that a pipe has been in service can be taken into consideration, namely, the coefficient ζ for new pipes being, as before mentioned, $0.019892 + \frac{0.00166573}{d}$, a new variable coefficient ζ_1 for any number of years of service may be approximately determined thus, $\zeta_1 = \left[\zeta + \left(\zeta \ \frac{1}{32} \ y\right)\right] = [\zeta + (\zeta \ 0.0313y)]$, y being the number of years of service.

The following table gives the discharge of a 48-in. pipe, having an inclination of 0.002, after it has been in service a specified number of years. The table was computed by using the variable coefficient ζ_1 in the formula $v = \sqrt{2gh} \div \sqrt{\zeta_1 \frac{l}{d}}$

Number of years in service.	ζ_1*	Discharge in gallons per 24 hours.
New.	0.02031	40 879 000
5	0.02349	38 012 000
10	0.02667	35 673 000
20	0.03302	32 060 000
50	0.05210	25 523 000
100	0.08388	20 115 000

* With the exception of $0.02031 = \zeta$.

Mr. Weston. The results given in the table appear reasonable. As can be seen by the table, the capacity of the pipe, owing to tuberculation, is reduced 38% after 50 years' service; and after 100 years' service 51%. or to the capacity of a new pipe about 36 ins. in diameter. A pipe 6 ins. in diameter under the same conditions would discharge when new 211 000 galls. per 24 hours, and after 100 years of service 104 000 galls. per 24 hours, or a reduction in capacity of 51% (as given above for the 48-in. pipe), which would be equivalent to the discharge of a new pipe about 4.5 ins. in diameter.

One of the best experimental illustrations of the reduction of the diameter of a pipe owing to tuberculation that the writer is familiar with is as follows:

The diameter of the pipe was determined with great accuracy before and after being cleaned, by filling the pipe with water and carefully measuring the same by means of a special apparatus arranged for the purpose. The average diameter of the pipe before the interior deposit was removed was 9.575 ins., and the average coefficient ζ was 0.0473. After the pipe was cleaned the average diameter was 9.634 ins., and the average coefficient ζ, 0.0271. The discharge of the pipe, at an inclination of 0.002, before it was cleaned, considering the above figures, would be 482 450 galls. per 24 hours, and after it was cleaned, 637 380 galls. per 24 hours.

A summary of these results shows that the coefficient ζ was increased by tuberculation 75%, the capacity of the pipe reduced 24.3%, and the diameter reduced 0.6%. Using the coefficient $\zeta = 0.0271$ of the cleaned pipe, in the formula before mentioned for determining the velocity, the inclination being the same, a clean pipe about 8.61 ins. in diameter would have the same capacity as the pipe 9.575 ins. in diameter before it was cleaned.

It may seem as though the writer was somewhat inconsistent in his suggestions relative to the variable coefficient ζ_1 as he has assumed as a coefficient ζ for new pipe one which was verified by experimental data of pipes of an age ranging from that of new pipes to those which had been in service from about two to thirteen years; and it might be thought that if the coefficient ζ_1 was increased owing to years of service, that the coefficient ζ used as a base should be assumed to be that of a pipe that had been in service a specified length of time, and therefore decrease in value for this specified length of time until the age of absolutely new pipe was reached.

If criticisms similar in purport to the above should be advanced, the writer's reply would be that Darcy's formula for new pipes had been proved to be a safe and reliable formula, and that experimental data obtained with a single diameter of pipe were not of sufficient magnitude for determining a correction to be applied to a formula which is intended to be used for pipes of all diameters. Also, that the variable

coefficient ζ_1 which takes into consideration the number of years that a pipe has been in service, is simply suggested as a reasonable approximation which has been based upon the only reliable data known to the writer, that have been derived from experiments made with the same pipes, when new and old, at an interval of time between the experiments which is positively known. It would hardly be possible to construct a reliable formula in which the length of service of a pipe was considered which could be generally used, as several important elements would enter into the case. For instance, the corrosion of the interior of an iron pipe is proportional to the volume of water flowing in it—the greater the volume, the more the corrosion; and the chemical constituents of the water exert an influence one way or the other, slightly alkaline and aerated waters causing cast-iron pipe to corrode much more quickly than some other waters differently constituted. Mr. Weston.

In closing, the writer would remark that in making use of a formula known to be safe for determining the size of supply or distribution pipe, the only criticism that an engineer is likely to receive is that his estimates of cost are too high. A criticism of this kind, however, generally helps the reputation of an engineer, as it is apt to give the impression that he is a safe man. The engineer, however, who tries to figure out the size of a pipe to the lowest possible limit on account of first cost, if the pipe does not come up to the capacity for which it was intended, is very likely to be blamed for having made a mistake, and his well-meant intentions in regard to first cost are, in the majority of cases, more likely to be ridiculed than praised.

George W. Rafter, M. Am. Soc. C. E.—The paper is of interest to the writer by reason of the well sustained conclusion that piezometer measurements accurately indicate the loss of head in a pipe line working under pressure, as well as for the light thrown upon the influence of the interior surface of pipes on the discharge. The vast importance of a protective coating that shall absolutely protect is forcibly presented. A number of years ago the writer studied the practice of American pipe foundries as regards the ordinary protective coating devised by Dr. R. Angus Smith, and came to the conclusion that at some of the foundries, at any rate, the coating was applied so carelessly that failure was fairly certain. As illustrating this point it may be mentioned that there are now cast-iron mains, laid in 1873, in the streets of Rochester, N. Y., in which the coating was about as perfect in 1890 and as free from blemish as when first laid; while pipe from other foundries laid several years later showed in 1890 very much the same appearance of the interior surfaces that is indicated in the illustrations in the paper. With the present knowledge the difference in such a case must be ascribed almost entirely to differences in quality of the material of the coating, or to defects of some sort in the method of applying it. Mr. Rafter.

Mr. Rafter. In the course of the study of coatings referred to, letters were addressed to several of the leading foundries asking for a detailed account of the formula in use at each foundry. The interesting point revealed was that in many cases the entire business of coating was a purely routine matter, which, from the foundryman's point of view, at any rate, was of relatively little importance. At one foundry at which the writer had occasion to spend considerable time there was no person with any knowledge whatever of the composition of the coating. A proprietary preparation purchased in large quantities was used without test and without question from anybody. So long as the coating hardened readily and adhered, it seemed to be assumed that every necessary condition had been fulfilled. The fact that the material used for producing the coating was a complex one, and hence necessarily subject to special conditions for the best results, was entirely overlooked.

Facts of the character of the foregoing lead easily to the conclusion that as regards the coating of water mains much information is needed. If the Boston water has a specially destructive effect on the coating, that fact should be strongly brought out by technical investigations, and, when established, a special coating devised for Boston mains to meet the emergency. In the same way the effect of other waters upon the ordinary coatings should be studied, and, if necessary, different preparations devised to meet the various cases. On this point it may be remarked that it seems very probable that the Boston waters are specially destructive in their action and some of the earlier information of the Boston Water-Works reports is interesting and valuable on this point.* The softness of the waters about Boston may be assigned as the chief reason for the specially destructive action.

The process of coating as ordinarily applied to cast-iron pipe was devised by Dr. R. Angus Smith, and first used on the mains of the Manchester, England, Water-Works about 1849 or 1850. Inasmuch as information about this very common operation is not easily obtained, the writer refers to it here.

According to Mr. Bateman† the process as originally used at Manchester consisted of immersing the pipes when hot in a bath or cauldron of boiling coal pitch after the naphtha compounds were distilled off. The residuum was then hard, insoluble, odorless and tasteless, and required an admixture of mineral oil to enable it to acquire, when boiled, proper fluidity for the operation.

The experience at Manchester demonstrated that if rusting, however slight, had taken place on the surface of the pipe, the coal pitch coating peeled off. The proving of the Manchester pipes was all done

* See "Annual Report of the Cochituate Water Board for 1852"; also "A History of the Introduction of Pure Water into the City of Boston," Boston, 1868, pp. 149 to 166.

† History and Description of the Manchester Water-Works. By J. F. La Trobe Bateman London and Manchester, 1884, pp. 143–146.

by the corporation before the application of the protective coating, and in order to prevent rusting the pipe foundries were required to paint the pipe with linseed oil, both inside and outside, as soon as the sand was cleaned off and while the pipe was yet warm. In this condition the pipe was delivered to the corporation for proving and coating. The proving consisted in subjecting them to a hydraulic pressure equal to 300 ft. head, after which they were heated in a vertical pan and then immersed in another containing the boiling coating, into which they were slowly lowered, still in a vertical position, and raised again. The heating drove off any slight rust or dampness and allowed the coating to penetrate the pores of the iron, which were slightly opened thereby. Mr. Rafter.

Mr. Bateman states that the resulting smooth surface added materially to the volume of water discharged. When the coating was fresh the increase over the quantity indicated by the ordinary formula was 40 to 50 per cent. He also states that an examination of some of the 36-in. pipes which were coated in this manner and had been laid 30 years at the date of examinations, showed that no corrosion had taken place inside, but that they were still as clean as when first laid. While at Manchester in October, 1894, the writer took occasion to inquire as to the present condition of the pipe first coated by Dr. Smith's process, and was informed that so far as the Water Department had information, derived from cut-outs, etc., they were still generally as clean as when first laid.

Mr. Bateman said further that where the oiling was neglected and the quality of the coal pitch not what it should be, the pipes had frequently to be discarded. He had, therefore, always insisted on the oiling previous to the coating. He also said that waters of more than 4 or 5 degrees of hardness precipitate an incrustation of lime on the interior of coated pipe the same as uncoated, which will in either case eventually choke the pipe.

The first use of pipe coated by Dr. Smith's process in the United States was pipe imported from Glasgow for the Brooklyn Water-Works in 1858.* The Brooklyn specification has been the basis of most of the pipe-coating specifications thus far used for cast-iron pipe in the United States. These specifications begin by stating that the conditions laid down must be strictly observed in order to insure the permanence of the coating and the efficient protection of the pipe from rusting. Some pipe foundries are not following this original specification closely which, so far as known, has not been improved thus far.

The original cast-iron mains of the Rochester Water-Works were coated under the original Brooklyn specification properly followed, and

* See "The Brooklyn Water-Works and Sewers, a Descriptive Memoir." New York, 1867, p. 41.

Mr. Rafter. it is believed from the examinations made by the writer five years ago that this coating is generally intact to this day. The wrought-iron conduit of the Rochester Water-Works was, however, coated by a quite different formula, derived, it is believed, from Californian experience. The following is the specification for this coating:

"*First.*—Every pipe must be clean and free from earth, sand, or rust, when the coating is applied, and no pipe shall be coated until the authorized inspector shall have carefully examined it in reference to compliance with the above requirement.

"*Second.*—The coating mixture shall be prepared and applied to the satisfaction of the chief engineer and in the following manner: The purest quality of asphaltum, such as shall be approved by the chief engineer, shall be procured and broken into pieces containing from two (2) to four (4) cu. ins. and placed in kettles over a heating furnace; then the interstices between the pieces are filled with the best quality of coal tar, free from oily substances, and the whole boiled and stirred up in such manner as the chief engineer may direct from three (3) to four (4) hours until the kettle charge is a semi-fluid mass. During the boiling such occasional tests shall be made as may be required by the chief engineer to determine the quality and character of the mixture and test its adaptability for the purpose intended.

"When the mixture is in proper condition it shall be drawn from the kettles into a reservoir over a heating furnace or oven of sufficient capacity to enable the pipes in lengths up to thirty (30) ft. to be thoroughly dipped.

"Immediately after being taken out of the coating bath, a piece of the coated iron is to be plunged into water near the freezing point, and if after removal the coating does not become brittle with a tendency to fly off or loosen when pipe is rapped with a hammer, but firmly adheres to the iron, the material will be considered of good quality, and the work properly done in ordinary cases, although the engineer may subject the same to such other tests as he may deem desirable.

"After the shorter pieces of pipe are riveted or bolted together and placed in a trench, a workman shall be sent through the pipe with a small kettle of the mixture and thoroughly coat the newly riveted joints, rivets and laps, and all other places where the coating has been marred, injured or destroyed in any manner. The same shall also be done on the outside of the said pipe, either before or after the laying in the trench or both, if required."

Recapitulating the subject of pipe coating it may be remarked:

First.—Mr. Bateman observed over 40 years ago that a smooth interior coating increased the delivery of a pipe when the coating was new from 40 to 50% more than indicated by the formula then in common use.

Second.—The maintenance of the integrity of the coating is a matter of supreme importance, and hair-splitting formulas are of absolutely no use so long as an indefinite reduction of the delivery is possible, due to a more or less constant deterioration of the interior coating.

Third.—As regards cast-iron water mains the coal pitch preparation of Dr. R. Angus Smith as originally applied at Manchester is the best thus far devised. So far as is definitely known it protects the pipe in-

definitely if applied strictly in accordance with the original specification. Mr. Rafter.

Fourth.—In view of the present knowledge of the matter, serious tuberculation of a pipe coated by Dr. Smith's formula within 25 or 30 years is probably presumptive evidence of improper or careless application of the coating. In drawing the fourth conclusion it is recognized, however, that coal pitch is a substance more or less subject to change, and there may be some waters which accelerate these changes more than others. On this general subject more information is needed.

Fifth.—This conclusion is stated in the form of a question. Is there any good reason why a coating should be used for wrought iron or steel pipe different from that used for cast iron?

In the writer's paper* on the "Hydraulics of the Hemlock Lake Conduit of the Rochester, N. Y., Water-Works," there is expressed a mild doubt as to the correctness of the value of c in the expression $v = c\sqrt{r\,s}$ as applied to pipes of long lengths and large diameters. Since the presentation of that paper in 1892, the views of hydraulic engineers have undergone some considerable clarification through the operation of forces to which it is unnecessary to refer specially. As regards coated cast-iron pipes of great length and large diameters the doubt is effectually removed so long as the integrity of the interior coating is effectually maintained. On the other hand, for built-up wrought-iron or steel lapped and riveted pipes the doubt has equally resolved itself into a certainty. It is known definitely that for such pipes large values of c in the Chezy formula are entirely inapplicable. Even when the pipes are new, the irregularity of the interior surfaces, caused by the projecting rivet heads as well as the lap of the sheets, introduces disturbing elements fatal to high efficiency.

In concluding the discussion of his paper on the "Hydraulics of the Hemlock Lake Conduit" the writer expressed the view that, as a matter of rational design, the built-up pipes are only applicable when by reason of high pressure or large diameter cast iron cannot be used safely. This view is strengthened by later developments. Certainly the assumed economy of the built-up pipes is considerably decreased by the necessary increase in diameter in order to compensate for the disturbing action of the rivets and the lap.

There is, moreover, one further consideration in favor of cast-iron pipes, namely, that cast iron is a material much less easily corroded than either wrought iron or steel, and hence, *per se*, a less difficult material to shield properly by the protective coating. It seems clear that a failure to appreciate this point has been the source of ultimate unnecessary expense to some of our municipalities where the built-up conduits have been used, and where they have been adopted on only a slight margin of first cost in their favor, and, so far as known, without regard to the final results.

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* See *Transactions*, Vol. XXVI, p. 13.

Mr. Herschel. CLEMENS HERSCHEL, M. Am. Soc. C. E.—If a meter were put into each of the 48-in. pipes described in the paper the record of the meters would show the gradual diminution of, possibly an occasional or regular or annual fluctuation in, the discharge of the two pipes.* As an effective meter for 48-in. pipes and for pipes of any diameter was described in the *Transactions*† in 1888, and has since become an established method of metering water, the opportunity was here presented of using such a meter, instead of the more costly, cumbersome and merely transient method of weir measurements. Indeed, it would not be surprising if the present set of experiments proved to be the last set of weir experiments ever made for the like purpose. In the progress of the arts there has to be a last time for all kinds of ways and methods; and in the nature of things there will have to be a last time when weir measurements are to be undertaken to meter the flow through a couple of 48-in. pipes forming part of a system of water supply.

In the *Journal*‡ of the New England Water-Works Association is an account of the test of a 48-in. Venturi meter, and its results are compared with those found in 1886 in the test of a 12-in. and a 108-in. meter of the same sort. These three tests show a conformity with each other not found in weir experiments. The theory of the Venturi meter also is simplicity itself compared with that of a discharge over a weir,§ and, beside this, various users of the meter have tested it for themselves, so that, to him who will enquire or study, the accuracy of a Venturi meter will speedily stand established. It is less likely to err than a weir, because it acts in conformity to simple instead of according to complex hydraulic laws, and both use and experiment show it to be exceedingly uniform in its results.

It costs much less than a weir in first cost, and still less to operate. The writer has endeavored to secure from the author of the paper the recorded cost of the apparatus described, and of the experiments conducted with it. In default of receiving this statement, the following estimate of an experienced foreman and millwright is given:

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* To illustrate. On March 2d, 1896, the State of New Jersey was visited by a cold, violent northwest wind, following a freshet that had removed the ice from rivers and ponds. This wind and cold lasted without interruption for more than 72 hours. The consequence was that the water-works of Camden, Trenton, the East Jersey Water Company, Paterson, Newark and Jersey City were all either shut off entirely or sadly diminished in carrying capacity by anchor ice. The 48-in. riveted steel conduit of the East Jersey Water Company, which has such a meter on it, had been carrying 36 000 000 galls. per 24 hours. During the period of anchor ice it carried about 23 000 000 galls. per 24 hours. After the scouring it got from ice that entered the gate-house through broken screens, and past screens lifted out of the way to keep the water running, it was found to carry 37 500 000 galls. This immediately suggests, as a safe and effective way of cleaning water pipe, to give them annual doses of cracked ice in liberal quantity.

† Volume xvii, p 228.

‡ Vol. viii, p. 35.

§ See on this point Merriman's "Treatise on Hydraulics," Art. 71.

Carpenter work at siphon chamber, Figs. 8 and 9.....	$444	Mr. Herschel.
Lumber.....	156	
Iron work.....	225	
Brass pipes.....	50	
Piezometer sleeves.....	500	
Piping	150	
Gauges and shelters.....	300	
	$1 825	
Weirs and gauges at terminal chamber.....	600	
	$2 425	

The time spent in taking observations and making computations must have been not less than five months of three gate-tenders and four observers and computers.

For a moderate estimate:

Three men at $60 per month.....	$900
Four " $75 "	1 500
Total expenditure for experiments.....	$4 825

Instead of this, a wooden Venturi meter with brass-lined cast-iron throat piece could have been built inside of the two 48-in. pipes, precisely like the meter described in the *Journal* of the New England Water-Works Association, and each one furnished with a register for less than half the money. The meters would have had the enormous advantage of continuing their record indefinitely, as long as the pipe lasted, and at a mere nominal expense of operation, instead of having to be pulled out, as had to be done with the weirs, and instead of requiring a large outlay for operation while the weirs remained.

The presence of a weir greatly disturbs the hydraulic gradient, as it necessitates the formation of a pond of still water up stream from the weir, and a depth of water flowing over the weir, on top of this pond, in addition; about 4½ ft. vertical fall in Fig. 17; whereas, a 4-ft. Venturi meter, made to meter the same range of velocities (0.5 to 7.25 ft. per second), and to keep the loss of head at a small amount, would have been made with a throat area of about two-ninths the full area of the pipe, and would, at 7.25 ft. velocity, have shown a loss of head in passing the meter of about 2.2 ft. The velocities of 5.5 and 3.6 ft. mentioned would have shown a loss of head of about 1 and 0.4 ft., respectively.

The difference between $c = 110$ and $c = 140$, or between the clean pipe and the pipe after 18 years' use, means a loss of head on 1 800 ft. length, due to tuberculation alone, when $v = 7.25$, 5.5 and 3.6 ft. per second, of 3, 1.7 and 0.7 ft. respectively, or of more than would be caused by a Venturi meter.

Mr. Herschel. On page 258 are given some "formulas chosen for the present investigation," but if the paper teaches anything it shows that the chart of logarithmic homologues had not been finished, and its derivative exponents had not been determined before the pipes experimented on had changed their carrying capacity sufficiently to render these exponents inapplicable. Looking back over the hydraulic literature of the past 30 or 35 years, it seems to the writer that he could readily count up at least twenty different sets of formulas set up during that time, each one as the best with which to compute the carrying capacity of pipe lines. Nearly each year sees such created, each one derived, first, from a penible fitting and swinging of plotted curves to represent as well as they may certain plotted experiments; and second, by means of one of the several mathematical sleights of hand devised to convert curves into formulas. All such formulas are delusive, because they portray only the discharge of the pipe or pipes in question at one certain period of their life; whereas, it is more important now to get the discharge of the same pipe, and of many kinds of pipe, at many periods of their life. The paper is perhaps unique in literature on the subject, as giving the discharge of the same pipe at an interval of many years, and nearly so as giving such discharge before and after cleaning.

It is true that to get the law of variation due to the different sizes and slopes of the same kind of pipes, it is necessary to compare them when they are new or smooth, but this seems to have been sufficiently attended to. Where a coefficient for the same pipe varies in 18 years from 110 to 140, the form of the formula becomes of minor importance, and the Chezy form will be considered by most engineers as good enough for all practical purposes, the coefficients being once closely determined. Possibly the use of the Kutter scale of roughness, or of improvements upon that scale, may eventually make it easier to find the coefficient by judgment alone, though the writings of engineers do not yet indicate that a scale of roughness is a materially better aid to finding true coefficients by judgment alone than a table of the coefficients themselves.

It is easy to adduce examples. Thus, Rudolph Hering, M. Am. Soc. C. E., in discussing the paper by George W. Rafter, M. Am. Soc. C. E., on the "Hydraulics of the Hemlock Lake Conduit," * takes Kutter's coefficient n, for 36-in. and 24-in. new riveted pipe as 0.011, while he takes it as 0.012 for new cast-iron pipe; when any tyro in 1896 can compute for himself that it should have been taken at least equal to 0.013 for new riveted pipe; and when every beginner also knows now that it is equal to 0.016 under some conditions of use for that kind of pipe. For a 4-ft. pipe laid on a slope of 2 in 1 000, the difference between $n = 0.011$ and $n = 0.012$ is the difference in the co-

* See *Transactions*, Vol. xxvi, p. 40.

efficient between 145 and 130, or a difference of 11.5% in the discharge; and the difference between $n = 0.011$ and $n = 0.016$ is the difference in the coefficient between 145 and 95, or a difference of more than 34% in the discharge. Mr. Herschel.

Mr. Hering had, in 1888, translated and extended Kutter's book, and was therefore presumably as well posted in 1892 on the use of the Kutter formula as anybody. He had Mr. Rafter's carefully written paper before him, which gave for the first time reliable gaugings of the Rochester conduit, but in spite of all that, the use of the Kutter coefficient could not prevent him from making what would have to be counted as an error due to negligence, except for the facts noted, that even in 1892 the construction and computation of the carrying capacity of large riveted conduits was yet a new thing. Engineers then or up to that time most conversant with long conduits of that kind, had united in opinion and in publications on the subject in treating it as of a similar degree of interior roughness as cast-iron pipe, and also as not subject to material deterioration of interior surface in the course of time like cast iron. So the record shows, that in spite of the use of the Kutter coefficient of roughness, an able engineer like Mr. Hering was, in 1892, legitimately under the same or a similar delusion with regard to the proper coefficient to apply in computing the carrying capacity of riveted pipe and with regard to the true carrying capacity of the Rochester conduit, as the record shows a long line of eminent and able engineers, also the writer, to have been, up to 1889 or 1890. Over and above all this, those who have themselves experimented on riveted conduits know that Kutter's coefficient of roughness does not compute a constant for one and the same pipe, during one and the same series of experiments. It varies sufficiently to cause the discharge to vary 25 or 30% from what it would be on the assumption of either of the two extremes of degree of roughness applicable to the same pipe at any one time, this roughness varying with the velocity. This is especially true in cases of a high range of velocities, and it is not yet properly determined when the coefficient c increases and when it decreases with the velocity.

Another such case is pointed out on page 257 of the paper now under discussion, by showing that the Kutter coefficient of roughness for tuberculated pipe varied with the velocity from 0.012 to 0.014, meaning a possible variation of discharge as 110 to 130, or 18%, according as the pipe be assumed to belong to one category of roughness or the other.

Another such case is found on page 141 of Mr. Hering's translation of Ganguillet and Kutter's "Flow of Water in Rivers and Other Channels," where the Kutter coefficient of roughness varies with the velocity from 0.0127 to 0.0160, meaning a possible variation of discharge

Mr. Herschel. as 112 to 142, or 27%, according as the pipe be assumed to have one degree of roughness or another.*

These cases should be sufficient to show the lack of support to be derived from a Kutter coefficient of roughness, in computing the discharge of a new kind of water channel. They should prove that a claim of increased accuracy to be derived uniformly from its use is in error, for a coefficient of roughness which represents the roughness of a pipe as changeable within a minute or a second to an extent of affecting the discharge 25% and over is a misuse of terms, and is itself no better than the plain coefficient it endeavors to replace. It is worse, because one variable coefficient is better than two. In other words, Kutter's coefficient, in many cases, does not apply, and for such is worthless. It was worthless for riveted pipe in 1889, and is yet, possibly, because Kutter had not had sufficient experiments on this kind of pipe at hand to use in constructing his empirical formula. It is better as an aid to an exhibition of afterthought, than it is to facilitate the exercise of prevision or of prophecy, unless used for old and tried cases, and such as were used in setting up the empirical formula.

The writer is at this time in position to make experiments on many kinds and sizes of riveted conduits. He is making such, as occasion demands, and has long intended to find their coefficients and to publish them in digested form, when somewhat complete. At present, the data available concerning this kind of conduit, so far as the writer has yet seen them published, are still too meager to permit of aught but ill-founded, possibly illusive, and mere ephemeral speculation and discussion.

Referring again to the exponential formulas of the paper, there is, in the writer's opinion, no utility, in the present state of the knowledge on this branch of hydraulics, in setting up specific formulas to represent specific experiments or gaugings. As a diversion from the line of advance on which such studies should be concentrated, there may even be in it somewhat of impropriety.

Toward the close of the paper is this sentence: "The author is particular in calling attention to this fact, on account of slurs that have been cast by some hydraulicians upon piezometric observations." It will take additional observations to those recorded in the paper to instruct an expectant profession on the value and the true meaning of this class of record readings. It has been observed, for instance, that piezometers do not always read alike, when tapped into a pipe at different points in the circumference of a pipe. This is noted on page 248, Vol. XVII of *Transactions*, and is also discussed in Merriman's "Treatise on Hydraulics," Articles 70 and 84.

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* The gauging here referred to is erroneously stated to have been made with a Venturi meter. It was made over weirs, of the water flowing in a tube subsequently fitted with a Venturi meter.

The opportunity existed during these experiments to renew the inquiry into this phenomenon, to note which it would only have been necessary to tap more than four piezometers into the pipe at the several stations, and to take observations on each tube separately. But by tapping four only and then connecting the lead pipes in pairs, the phenomenon, if it existed in this case, must have been obscured beyond recognition. Mr. Herschel.

The piezometers described in the paper were 1 in. in diameter, which is an unusually large size. The writer introduced the method of leading the orifices from the pipe into a pressure chamber, and then attaching the gauge tube to the pressure chamber, and this method has given good results in the hands of other experimenters, but should be further experimented on.

It certainly is not clear to-day, any more than it ever was, whether piezometer readings are wholly a function of the mean velocity, or partly, at least, of the surface, or some other internal velocity of the water contained in the pipe also, or how they are affected by position, angular or otherwise, or perhaps by size, or just what is the best and hydraulically most accurate and perfect way of constructing and placing them, or what should be the standard way of constructing and placing them. So that it is fair to assume, that much still remains to be sought and discovered of matters relating to piezometers.

Desmond FitzGerald, M. Am. Soc. C. E.—The only portions of the discussions which call for serious attention are those which attack the propriety of the exponential form of equation submitted by the author. Mr. Hering champions the Kutter formula warmly, as might have been expected from his long and able work in connection with its use by American engineers. Mr. FitzGerald

The value of Kutter's work is appreciated by the author, who was careful to give the results of his experiments in the Kutter as well as the exponential form. If, however, it should appear from recent gaugings that the monomial form of equation, with variable exponents, is capable of a wide and accurate application, and that it is both simpler and easier to use, the author sees no reason for clinging blindly to any other formula, no matter what its history may have been. It is known now that the Chezy formula upon which Kutter's is built is founded upon a serious error. Resistances do not vary exactly as the square of the velocity, however much the text books may insist to the contrary.

Mr. Hering believes, however, that the "only" proper course to pursue is to start with the Chezy formula. The difficulties met with will be apparent upon a close perusal of Mr. Hering's graphic description.

Mr. FitzGerald As early as 1850 M. Barré de Saint-Venant* proposed a formula with two arbitrary constants which he attempted to ascertain. From the data then existing he found the index of the velocity to which the resistances were proportional to be 1.71 instead of 2.0 in the Chezy formula. It is known now that 1.71 is a very low index and represents an extraordinary degree of smoothness in the channel, corresponding to tin plate.

Is it an "impropriety" to turn backward to the exponential form? The future must of course determine this. Its introduction in the paper was done at least after long deliberation, and before condemning it the author asks the student of hydraulics to study carefully the works of Saint-Venant, Hagen, Reynolds, Unwin and Foss. The last has shown that a number of trustworthy gaugings of cast-iron pipes give 1.88 as the exponent fitting closely the experiments on clean pipes, whatever their size or inclination.

The author found 1.91 in his experiments, and Prof. Unwin deduced 1.95 from his examinations. If it be true that a coefficient of roughness can be found which is constant for any given surface, whatever the slope, size and shape of the channel, then a valuable ally has been found whose acquaintance it is important to cultivate. It seems to the author that it must be as easy for an engineer to use a monomial formula with an exponent which can be varied to suit the varying conditions of roughness of channel as to apply the more complicated method of Kutter.

If Mr. Herschel's optimistic statements in regard to the Venturi meter are correct, then he is to be congratulated upon this remarkable revolution which has already relegated the weir to obscurity. Still the author is not persuaded that he would have done better to use the meter in these particular experiments. It would have been impossible to carry the velocities to over 7 ft. per second, as was done with the arrangement of weirs, and perhaps it might have been desirable to rate the meter by means of the weir. As far as the cost of the experiments is concerned, the author wrote Mr. Herschel that it would have been furnished to him had it been possible without much time and trouble. The work was carried on almost entirely by the regular maintenance force, and there seemed no sufficient reason to justify an investigation into the expense.

The author hopes that whatever this may have been, the results may prove of value to the profession. Mr. Gould seems to be uncertain on this point. The author is very sorry that the 48-in. pipes discharged so much more water than Mr. Gould seems to think they should have done. It is to be hoped that notwithstanding that gentleman's faith in the Darcy formula, he will not worry himself very much over fearing

* *Comptes Rendus Hebdomadaires des Séances de l'Académie des Sciences*, Paris, XXXI, 1850, pp. 283–6, 581–3; *Annales des Mines*, XX, 1851 (4me Series), Vol. 61, running number.

"that dangerous generalizations may arise from these experiments." Mr. FitzGerald
Heights of the water in the gate chambers were properly recorded and will be furnished Mr. Gould on application.

Mr. Herschel says:

"If the paper teaches anything it shows that the chart of logarithmic homologues had not been finished and its derivative exponents had not been determined before the pipes experimented on had changed their carrying capacity sufficiently to render those exponents inapplicable."

This is rather a formidable way of saying that if the flow through a pipe is measured, the calculations are hardly cold before the carrying capacity of the pipe has diminished by tuberculation. The value of this remark lies in its application.

In answer to the criticism upon the piezometers, the author can only ask Mr. Herschel to study the agreement between the readings of the piezometers, whether deduced from the tube piezometers lying flat upon the bottoms of the pipes or those screwed into the sides of the pipes. It was no part of the problem to determine the effect of piezometers set at angles to the flow of the stream, which has already been done by other experimenters.

Mr. Weston defends the Darcy formula with ability. As long as that formula gives results upon the safe side no one is hurt, but is it desirable to apply a formula to the discharge of a large clean main which really gives the discharge of an old and tuberculated main? Is it not misleading and liable to obscure the important fact that the carrying capacity of a pipe decreases rapidly with the changes which take place in the nature of the wetted perimeter as the pipe ages?

The following information is in answer to questions asked by Mr. Hawks. It is true that the circle of tubercles disclosed by the photograph is at a joint. The author, however, does not think that the joints generally showed this effect. Some of the tubercles were half an inch in height, but no blistering was noticed. The author does not know how well a rattler would work in removing tubercles.

A volume might be written upon the subjects suggested by Mr. Rafter. It is true that the coal tar used in coating cast-iron pipes is very carelessly applied in some foundries. There is ample room for reform in this direction. Much more attention should be given to the lining or coating of pipes. In the case of large mains the author has frequently noticed great ridges or concentric rings in the interiors and he has some fine photographs which show this common fault. The rings are caused by the impressions made in the soft clay cores by the process of leading, where the leading is done by hand with a large brush. The general lack of smoothness in the interiors of mains and the careless coating of the pipes after they are cast are ex-

Mr. FitzGerald pensive additions to the frictional resistances and consequent losses of head.

An experiment recently made by the author on a new 36-in. main a mile in length gave $v = 136\ (R\ I)^{\frac{1}{2}}$ for a velocity of about 4.7 ft. per second. It forms part of a series of measurements on this pipe at different velocities.

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No. 777.

THE DISCHARGE OF THE MISSISSIPPI RIVER.—CORRESPONDENCE ON PAPER No. 765.*

By F. P. SPALDING, FOSTER CROWELL, T. G. DABNEY, B. M. HARROD, W. G. PRICE, CHARLES H. MILLER, H. ST. L. COPPÉE, J. B. JOHNSON, ARTHUR HIDER, C. McD. TOWNSEND and WILLIAM STARLING.

F. P. SPALDING, M. Am. Soc. C. E.—It must be apparent to any one who has given much attention to the phenomena presented by the flow of the Mississippi River that the conditions are so variable and the data apparently so discordant as to make any attempt to reduce them to order or to deduce any general laws concerning the flow an almost impossible task. Undoubtedly the laws governing the flow of water are the same for the Mississippi as for any other stream. This, however, does not imply that the formulas for flow of streams as deduced from observations on a small scale are even approximately applicable to this case. Mr. Spalding.

Much time has been wasted in vain attempts to fit the ordinary formula $v = c\sqrt{rs}$, with a coefficient which shall make it applicable in all cases. Notwithstanding the great value to hydraulic science of

* "The Discharge of the Mississippi River," by William Starling, M. Am. Soc. C. E., *Transactions*, Vol. xxxiv, p. 347.

Mr. Spalding. many of these labors, and notwithstanding the value of the formulas themselves for use in general hydraulic work, they still are quite useless when applied to the Mississippi, and, in general, would be more accurate and better for general use if the Mississippi gaugings were not included in their derivations. The common formula $v = c\sqrt{rs}$ is based upon the assumption of a uniform flow at the section considered; that is, upon the absence of any accelerating force, which requires that the water shall have flowed for a sufficient distance in a channel of uniform section to attain a state of equilibrium between the accelerating and retarding forces, a condition which is probably very rarely, if ever, approached by the Mississippi.

The value of s in this formula is necessarily that of the surface in the immediate vicinity of the section under consideration, if the flow is to be uniform under the action of forces due to that slope at the instant of passing the section. The use of this slope in computing discharge by the formula often gives the wildest and most unsatisfactory results. In some cases, as stated in the paper, the slopes may even be negative, giving by the calculation a flow in the opposite direction from that actually occurring.

The use of any formula involving slope for so large a stream is apt to be misleading and unsatisfactory.

The author suggests a method of estimating flow from gauge heights, and it seems reasonable to suppose that an approximate idea may be formed in this way of the probable quantity of water which may pass at a given gauge reading, or, on the other hand, of the probable height on the gauge which will be due to a given flow. This method is certainly preferable to the slope formula, but it is to be remembered that these estimates at best are very rough approximations, and that individual instances will differ widely from the averages used. Whether the river is rising or falling, the duration of the flood stage and the amount of silt carried will affect the result.

The determination of the height to which the water in the lower river would rise if it were confined is therefore a very difficult problem, and the estimates which have been announced for probable maximum flood height under such circumstances are all to be taken with a grain of salt. The Mississippi has a way of adjusting itself to circumstances and of modifying its bed to suit changed conditions which make it reasonable to believe that the estimates quoted in the paper

may be somewhat excessive, and that the effect of entirely controlling the floods would be a smaller increase in flood height. Mr. Spalding.

In regard to the matter of rating meters, it is at least doubtful whether any additional accuracy is secured by abandoning the straight-line formula for ratings and using the curve. The fact that the ratings would form a curve when plotted is not a new discovery, as many observers have been in the habit of plotting them for a number of years, although using the straight-line formula. The straight line may, for the range of velocities commonly met in practice, be made to fit the ratings within the limits of the ordinary errors of observation, and thus introduces no appreciable error into the results. Increased accuracy in discharge measurements must be obtained, if obtained at all, by improving the methods of taking the observations, and not by introducing greater refinements in computation.

Foster Crowell, M. Am. Soc. C. E.—With such a voluminous paper as that presented by the author, it is difficult to single out parts for brief discussion. There are, however, three points which should be emphasized. The first is the inapplicability of the Humphreys and Abbot formula to general use in the gauging of rivers. The author clearly states the reasons for this—reasons which do not appear to have been generally understood by hydraulicians. For instance, Ganguillet and Kutter remark of it: "The American formula is specially adapted to streams with a gentle slope, and is not to be recommended for general application. Nevertheless, it is the outcome of most important studies, and will always retain its high value in its own field." As a matter of fact, it is of value only when used precisely in the conditions it was framed to suit. But for the erroneous assumption to which the author refers, it would correctly represent a particular case. It has already been demonstrated that it gives velocities entirely too small for streams of steep slope, and for streams of flat slope its results vary with the hydraulic mean depth, probably approximating nearest to correctness when the latter is about sixteen. The second point is in regard to the inaccuracy and unreliability of the forms of current meter that have heretofore been employed. They are, as the author has stated, inaccurate, both as machines and in their use. The writer would point out in this connection the importance of selecting properly the gauging places in a stream, so as to avoid the sources of error contained in deflected currents, for which purpose a Mr. Crowell.

Mr. Crowell. separate set of experiments upon the direction of local currents at different depths should be made before the current is metered. The third point is in regard to the effect of viscosity upon the flow of muddy streams, to which the author appears to attach slight importance. In the writer's opinion, here is a fertile field for important investigation, and he hopes at some future day to be able to contribute some results of researches in that direction. Between pure water and thick mud there would be a wide range of velocity for a given slope, and the effect produced by the presence of even a small proportion of muddy matter cannot be disregarded. This question is intimately connected with the movement of sand and gravel, which movement requires work to be done by the stream, reducing its energy and velocity.

Mr. Dabney. T. G. DABNEY, Esq.*—The subject discussed by the author is of the utmost importance to engineers engaged in the practical solution of the Mississippi River problem, and the paper is a most valuable literary contribution to the efforts in the field. It represents the results of a great degree of skill and industrious research in the treatment of a vast collection of heterogeneous data, full of anomalies, real or apparent, from which chaotic store of materials rational laws and valuable practical conclusions have been extracted with great ingenuity and patience. It may be further said that the paper presents in thoroughly digested form that which is of practical value in the voluminous and more or less confused records of observations of Mississippi River phenomena, placing it in available shape for convenient use by river engineers. The principal object of this laborious investigation is embodied in the following paragraph from the paper:

"The immediate object is to find out, first, what was the greatest actual discharge that ever passed down the Mississippi; * * * and, second, what height would such a flood have reached on the gauges of the several stations if it had all been restrained between banks and levees. Better than all would it be to find out the general law regulating the relations of the discharge to the gauge, so that it would be possible, not only to ascertain the height corresponding to the maximum discharge, but to any discharge, at any time, past, present or future. This last quest is the philosopher's stone of the river engineer."

In summing up his final results, the author recognizes the futility of attempting a complete answer to a problem so full of complexities,

* Chief Engineer Yazoo-Mississippi Delta Levee District.

and contents himself with a determination of the maximum discharge at a single station, Arkansas City, as being 1 735 000 cu. ft. per second, and the year of maximum discharge as being 1882, with the extreme gauge height which such a discharge would have produced at that station as being 55.8 ft. if confined between levees. The admitted imperfections in the methods and appliances available for making discharge measurements in such a stream as the Mississippi River forbids the acceptance of actual figures with entire confidence, but it is probable that comparative results are in the main correct, and that the conclusion as to the period of greatest discharge may be relied upon as true. What is of much more practical importance, it is probable that the conclusion regarding the gauge reading for the maximum discharge is correct within a narrow margin. The writer is inclined to believe that this conclusion is rather under than over the truth. Mr. Dabney.

There is only one position taken by the author in his discussion of Mississippi River phenomena upon which the writer ventures to offer a criticism. The author assumes* an alternative scour in the pools and fill on the bars at high stages of the river, with a reversal of these conditions at low or on a falling water. While this is true, it fails to state the whole truth. It is a well-known fact that very large quantities of material are deposited on the bars during high water, but it is a mistake to suppose more than a small proportion of it comes from the bottom of the pools. The river at high stages loads itself to its maximum carrying capacity with material taken from the caving banks in the concave bends, and this would seem to be the source from which by far the greater part of the mass which constitutes the high-water fill on the bars is derived. It follows then, if this be true, that the prevention of bank caving would result in a greatly diminished deposition on the bars, simply from lack of material carried by the current. It is true that wherever caving is checked, either by artifical protection or natural resisting powers of the river bank itself, the suspension of the lateral movement of the channel is accompanied by a transference of current energy from the side to the bed of the river, resulting in vigorous vertical scour. This movement, however, soon reaches its limit, and the quantity of material eroded from the river bed is insignificant as compared with the enormous masses dislodged from rapidly caving banks. The principal efficacy of bank protection, as a means

* See *Transactions*, Vol. xxxiv, p. 428.

Mr. Dabney. of regulating the river's regimen and correcting its numerous evil features, depends upon these conditions.

A large diminution of the material available for deposit on the bars should at once result in a marked increase of high-water depths, a result which ought to be promoted still further by the capacity for erosion of a volume of water with high-water energy and a deficient load, suggesting the probability of an increase of the bar depths by direct as well as negative agency. These conditions should result in a general tendency toward diminishing the disparity between the depths on the bars and in the pools at all stages, with the manifest results of improved navigation facilities and a lower high-water plane.

Mr. Harrod. B. M. Harrod, M. Am. Soc. C. E.—The paper is a full and fair exposition of the difficulties of river discharge measurements on a large scale. Probably these are greater on the Mississippi than on any other stream where gaugings have been attempted; at any rate, as observations of this character have been continued there longer and more systematically than elsewhere, the difficulties have been more fully disclosed, both as regards natural conditions and the methods and apparatus employed.

This river has a width below Cairo varying from 2 500 to 6 000 ft., omitting extremes. Its low-water depth is from 6 to 50 ft. at the bars and pools respectively, and its range from low to high water is 40 or 50 ft. Velocities of 15 ft. per second have been measured. Its curvature is very great, even for an alluvial stream, giving a midstream distance of 960 miles from Cairo to New Orleans, while the direct distance is less than 500 miles; consequently the flow is indirect, crossing about 175 times, from bank to bank, in this distance. These crossings may, at low stages, be nearly at right angles to the trace of the stream. There are vertical eddies, termed boils or whirls according as their movement is upwards or downwards, which have sufficient energy in extreme cases to raise or depress the surface about a foot. The current carries in suspension sand of various degrees of coarseness, small roots and other trash, and on the surface, drift, which is very heavy at the higher rising stages.

The difficulties of gauging the discharge are due to the magnitude of the stream and its turbulence, the horizontal and vertical indirectness of the currents, the rapidity of its changes, and the sand and foreign substances which it transports.

Mr. Harrod.

Evidently the accuracy of a discharge observation increases with the number of stations on the selected section, and the length of time each of these stations can be occupied. On the other hand, since the stage, cross section, slope, velocity, and consequently the discharge, may be constantly and rapidly changing, the observation should be concluded in the least possible time. It seems reasonable under these conflicting demands, that for the best results an entire discharge observation should be accomplished during a single working day. Within this limit, all possible detail should be crowded.

Discharge observations have been of two classes on the Mississippi. Whenever certain conditions, generally of high or low water, occur, which promise important results, local observations are made along those parts of the river where the greatest amount of work is being done, or where the greatest changes are expected. Again, a series of stations have been selected on several occasions for occupation for an entire year. This latter system would give the best results under favorable conditions, because the entire movement from low to high water and back again, over the whole river, would be observed by specially organized and practiced parties using similar instruments and methods. As such an organization must be determined on some time in advance, and as its cost is from $75 000 to $100 000, and as the year selected may not prove eventful in the sense of there being extremely high or low water, the undertaking involves risks and therefore has not been repeated as often as is desirable.

The discussion between double floats, rods and meters is still open, and the writer does not propose to close it. On the Mississippi, the merits, defects, and particularly the limitations of the two former devices, are well ascertained. While their demands on the intelligence of the observer are less than those of the meter, they are not less on his care and conscientiousness. With double floats not only may the sub-float be raised several feet, as noticed in the paper, but the surface float may be submerged, as the vertical eddy in which they are involved is moving upward or downward. The ways of the lower float may be as devious and obscure as those of the meter, giving all sorts of relations to its surface companion. With the rod the small part of the depth which its limited length can integrate, and the doubtful and varying relations between velocities and depths leave much uncertainty in its results.

Mr. Harrod. These two instruments are so simple and have been in use for so long a time without material change that it is hard to see how they can be made more useful on the Mississippi. While the meter is much more complicated, its evolution is of recent date and is still going on. The exigencies of the Mississippi River service have led to a form which withstands hard usage, has bearings protected against sand and grit, and is not readily encumbered by small roots, leaves and other trash, but is still highly sensitive. It certainly requires great care and skilful manipulation. Its failure to record the direction of currents as well as their velocities is a serious defect, which is, however, shared in a degree by both double floats and rods. Should the direction meter which is now being introduced prove adaptable to the conditions in the Mississippi River, it would appear that there is hope for progress in the art of gauging great streams in the perfecting of this instrument and its expert use.

It seems probable that greater use of the method of integration, both vertically and transversely, would detect movements which are likely to escape the observations ordinarily made, and eliminate errors which may be involved in an assumption of fixed relations between velocities at different points in a cross-section.

The part of the subject discussed on pages 420 and 422* of the paper is of extreme interest. Selecting the year 1887, "because it was not a year of very great flood, nor, therefore, of great disturbance of high-water conditions, and because it was a year of very low water," and also because sufficient levee building had not then been done to affect materially the characteristics impressed on the river during its long unleveed condition, the range from low to high water at the mouths of the tributaries and at intermediate points is found to be as follows:

Cairo	46.35	Mouth of the Ohio River.
Belmont	40.99	
New Madrid	37.14	
Fulton	32.38	
Memphis	34.10	
Helena	46.15	Mouth of the St. Francis River.
Mouth White River	44.25	
Arkansas City	46.35	First gauge below the Arkansas River.
Greenville	38.95	
Lake Providence	36.48	
Vicksburg	48.61	Mouth of the Yazoo River.
Natchez	43.45	
Mouth Red River	42.53	

* Vol. xxxiv, *Transactions.*

Bars with 6 to 8 ft. of water recur with sufficient frequency to make the low-water surface conform to the general shape of the bed. In plotting the high and low-water surfaces, it is clearly observable that the former present a more regular curve, and that the irregularities of the latter consist of depressions at and about the mouths of the tributaries, with elevations at intermediate points. Mr. Harrod.

If the beds of sedimentary streams are self-regulated, as is stated by the author and generally accepted by engineers, then the variations of discharge between parts of the river where it is augmented by the volume of the tributaries and the return of the overflow, and those parts where it is depleted by escape into the basins, may be reasonably accepted as a cause of the depression of bed where the discharge has been habitually increased, and of the elevation of bed where loss by overflow has been unchecked for a long period of time. If this relation of cause and effect exists, then, if the discharge is confined to the bed and not allowed to escape over the banks, and is thus made uniform throughout the length of the river, with the exception of the additions from the drainage from the tributaries, the bed should become quite uniform in depth and cross-section, with an increase below each tributary, proportioned to its discharge.

The confinement of the high-water discharge to the bed is now being effected by levees on some parts of the river. These artificial banks have not yet the height necessary to restrain the greatest floods, but since 1891 they have been able to hold them at a considerably greater height than ever before. It is important to ascertain if this increased restraint of flood discharge is producing the effect anticipated. At present there is no conclusive evidence on this question, but the indications are that the deposits between tributaries are being removed by the greater uniformity of flood discharge caused by the prevention of overflow. Where levees have been improved and the flood discharge increased, its height is not proportionately raised. The effect, so far, seems to be divided between a higher surface and a lower bed.

If a general summary is made of the gauge records from 1882 to 1894 inclusive, they can be classified, as regards time, into two divisions, from 1882 to 1890, and from 1891 to 1894. They may also be divided locally, from Cairo to Helena, and from the White to the Red River. This division in time agrees substantially with the beginning

Mr. Harrod. of effective levee progress, and the division in locality is between those parts of the river which are unleveed or leveed on both sides. If, with these divisions in mind, the mean of the low waters over the entire period, from 1882 to 1894 is considered, it will be observed that for the years since 1890 they were lower through the entire river below Cairo than in the earlier years; but it will be observed further that while the mean relative depression where no effective extension of the levee system is yet made, or from Cairo to Helena, is 2.89 ft., from the White to the Red River, where greater levee progress has been made, the depression is 4.46 ft., or 1.57 ft. more.

If the mean of the low waters of the years previous to 1891 are compared with the low water of 1894, it appears that while the relative depression in the latter year in the upper or unleveed part of the river was 4.88 ft., in the lower or leveed part it was 7.41 ft., or 2.53 ft. more.

The period during which the river has been the subject of observation and work is too short to give definite conclusions, but the tendency shown in the foregoing statement, and by other indications, is quite marked for the limited time considered.

Mr. Price. W. G. Price, M. Am. Soc. C. E.—In the work of measuring the discharge of a great river most of the errors are probably due to eddies and cross-currents, and to a lack of skill on the part of the observer. Discharge sections should be located where the flow is most uniform. A section near the lower end of a straight reach will generally give better results than one just below a bend. Much inaccurate work has been due to imperfect meter ratings. After a meter was rated, the observer had no way of testing his work, except to rate it again, and, owing to a lack of skill, no two ratings were alike. There is a method of comparing meter ratings which is a practical test of the accuracy of the work, and is as follows: All the ratings of a meter, when reduced to revolutions of the wheel per second, and velocity in feet per second, and plotted on co-ordinate paper, should give parallel curves. The distance these curves lie apart depends entirely upon the friction of the bearings of the meter wheel at the time the ratings were made. When a meter has been rated so that the curve has been accurately determined, all subsequent ratings should give the same curve or parallel curves, unless the meter has been altered in shape. If the friction of the bearings only has changed, the curves will lie apart from each other, but they will remain parallel. A slight change in the

shape of a meter wheel makes only a very slight change in the curve. Mr. Price. Accurate ratings, made by the writer at different places and by different methods, of seven meters of the same size and kind, gave curves which were as nearly parallel as they could be plotted. The distance from the origin at which the rating curve intersects the axes of ordinates represents the friction of the bearings, and by trial in still water the wheel will begin to turn when the meter is moving at a velocity only a trifle greater than the intercept on the axis of ordinates obtained by a careful rating.

Several ratings of meters on the lower Mississippi, which were reduced on the assumption that a rating could be expressed by the equation $y = ax + b$, have determined b to be a minus quantity. If this result were true, these meters would solve the problem of perpetual motion. Such a result with any meter, or by any method of computation, shows the work of the observer to be inaccurate. Slight changes in the friction of the bearings make but very slight changes in the value of b.

The experience of the writer indicates as untrue the theory that when a meter is cleaned or a bearing oiled the effect is immediately perceptible in the rating. Real variations in the correct rating are caused by serious injury to the meter, or by it being out of order in some way that can be seen easily by a skilful observer; and when it has been properly repaired, the rating will be practically the same as before. It is not subject to constant changes in rate when properly cared for, but these apparent changes are due to a variation of the currents which are allowed to act on the wheel in rating it, and to other errors of observation. Some engineers lack the mechanical skill which is needed to know when the meter is in good order. It is very necessary for the lower bearings of meters like those in use on the lower Mississippi to be in such condition that the current, pressing against the wheel in a direction normal to the shaft, will not cause the wheel to rise in its bearings, as if it does so, the lower air tube is liable to come in contact with the base of the pintle, which will greatly increase the friction. The meter should be tested by pressing the hand against the wheel in the direction normal to the shaft.

The meters which have been most used on the lower Mississippi are so constructed that it is impossible for sand or grit of any kind to get into the bearings, and when they are well cared for the friction is a constant quantity.

Mr. Price. The rating curve of these meters, which are necessarily strong and heavy, departs more from a straight line than the rating curve of the acoustic meter, which is much lighter in construction and has much less friction in its bearings, but the Ellis meter, with a mercury cup contact breaker attached to it, which makes the friction the same as that of the heavy meters, also gives a curve approaching as near to a straight line as that of the acoustic meter. The difference in the curvature of the ratings of different kinds of meters is evidently due to the forms of the wheels, and not to a difference in the friction of the bearings. The method of rating described in *Engineering News* of January 10th, 1895, will enable the observer to secure more accurate data than can be obtained by some methods which have been commonly used. It eliminates some sources of error, and enables a skilful observer to determine the direction of the curve and the intercept very closely.

The curves of vertical velocity for different depths and stages of the Mississippi have not been as accurately determined as they should be. Some of the observations for this purpose have been made with apparatus which could not give accurate results. Those made in 1882 were inaccurate for two reasons at least. The meter had bearings that soon became filled with sand, which changed its rate, and with the apparatus used to suspend the meter in a deep swift current it was impossible to tell at times within several feet at what depth the meter was measuring the current. At Paducah, in 1882, and at Carrollton, in 1883, a better meter was used, and a different apparatus for suspending the meter enabled the observer to locate it very closely at all depths.

At Sioux City in the winter of 1894 the vertical velocity curve in the Mississippi River under the ice was very carefully measured by the writer by a series of observations. The percentage of mean velocity to mid-depth velocity varied, for different parts of the discharge section, from 0.901 to 0.915. The mean of all observations was 0.91.

Mr. Miller. CHARLES H. MILLER, Esq.—Many of the difficulties encountered in observations on the Mississippi are due to the choice of location. The general practice thus far has been to locate the section on a narrow part of the river, a straight reach if possible, but this does not receive as much consideration as the width. Instead, the section should be

located on a wide, straight reach, where the currents are flowing nearly parallel to each other. The narrow river is under abnormal conditions; the water, being forced in at the head of the reach at various angles, causes boils and eddies, and the increase of depth below datum over the shallow section indicates a vertical current, especially when the section is in the upper end of the pool, as at Arkansas City. Mr. Miller.

The main objection to a wide section would be the length of time consumed in taking an observation, permitting the currents to change considerably before the observation could be completed. It is a question, however, whether or not there are any very great changes in such a section. Nevertheless, the trouble could be easily obviated by an increase of force and apparatus; in fact, the ideal condition would be to have a line of current meters extending across the river, all observing at the same time. The time of taking an observation has, in recent years, been reduced fully one-half. The method formerly pursued was to take velocity observations over the entire section first, which required from two to three hours; then the section was sounded to obtain the area. Very often the interval between these observations was as much as three to four hours. The method now used is to measure the section at the same time that the velocities are taken, sounding between stations as the steamer moves from one to another. The time of holding on stations has been lessened, and is usually two minutes instead of from five to fifteen. This reduction has not impaired the accuracy of the work at each individual station, and, taking them collectively, it has reduced the chances for changes in the currents.

Regarding the difficulties of manipulation, a thorough examination of the error due to the drift of the steamer was made during the observations of 1892. The path of the steamer was found by two transits on the shore, taking simultaneous readings every 30 seconds to a pole placed directly above the meter. These points were plotted to a large scale and the required computations made, showing practically no error due to lateral drift, and very little to direct drift, the sum rarely going over ½%, and nearly all of it found on one or two stations. Greater care is now taken that the steamer is on the range at the times of beginning and ending, thus eliminating direct drift also.

A 3-ft. vane on the lower end of the lead-weight holding the vertical rod on which the meter is placed precludes any error due to the meter being forced into any other than a horizontal position, except

Mr. Miller. by the currents themselves. This was determined by fixing in the side of the weight an automatic appliance for measuring the greatest angle of inclination, and was used for depths as great as 60 ft. without finding any appreciable error. It may be mentioned here that this weight is of lead, presenting a wedge-shaped face to the water fore and aft; it weighs 250 lbs., and is lowered with a ¼-in. steel cable from a drum. A guy line of No. 12 wire attached to the top of the rod, on which the weight is fixed, and leading up to the forward end of the steamer, prevents the current from carrying the weight and meter back of a perpendicular. Tests under the above conditions have proved that the meter can be placed within 0.5 ft. of the desired position.

As to the discrepant results shown by the observations at Arkansas City and Wilson's Point for 1891 and 1892, and the near agreement of the results for 1893 at the same places, it is evident from the thorough tests of 1892 that the disagreements were not due to errors of observation. Among other tests, the meters were exchanged, each observer taking the other meter as a new one, rerating it, and applying the rate found to his observations taken with it. Next, the chief observers were changed, each taking the other party and section, and finally the Wilson's Point party and outfit moved to Arkansas City and worked side by side with the other party on that and the Chicot City ranges. Still the results accorded from day to day. The only reasonable solution of the difficulty seems to be the fact that the positions of the ranges for 1893 were changed somewhat from the old locations of 1891 and 1892, in order to get the ranges more nearly perpendicular to the flow, and also at Wilson's Point to escape as much as possible the eddy action on the Mississippi side. In the "Report of the Mississippi River Commission for 1894," it is stated on page 2813, for Arkansas City:

"The old range used for several years past was changed; in azimuth swung down stream 6° 40′, its east end being 560 ft., its west 200 ft., below the old position."

For Wilson's Point, on page 2816, it is said:

"As shown on the map which accompanies this report, the section was moved about 250 ft. below the range used in previous years; first, to try and get below the eddy on the Mississippi side; and, second, in order to have the range just 2 000 ft. below the transit point ⅃ Ruple. The direction was also changed about 4½°, in order to have it more nearly perpendicular to the flow, as evidenced by the flow across the section last season."

H. St. L. Coppée, M. Am. Soc. C. E.—The vast heterogeneous accumulation of data in the reports of the Mississippi River Commission has remained practically undeveloped or arranged, though explored by many for special purposes. By one who is accustomed or compelled to delve in the several thousands of pages of these volumes, the difficulty of the task undertaken by the author is fully appreciated. The investigation relative to the physical phenomena of the Mississippi River instituted and being carried on by the Mississippi River Commission is an engineering work second in importance to none that has been undertaken during the present century. Mr. Coppée.

It is undoubtedly true that the fundamental laws of hydraulics as applied to rivers or streams are "called into question at the present day," by engineers and authors of reputation. The important facts and laws of physical science have been discovered by experiment or concrete physical evidence, and not by abstract deduction from hypotheses originating in the brain of the mathematician. By this great work of the Commission sufficient data may be obtained in a time not far distant to direct the way, at least, to greater discoveries in the field of hydraulics, and lead to more perfect methods for the improvement of other streams as well as the Mississippi.

As the author states, the difficulty of ascertaining the discharge of a river like the Mississippi is very great. Indeed, it cannot be obtained free from considerable error, owing to the presence of inequalities in the bed and cross-section, and other phenomena causing very irregular flow to which no simple formula will apply and for which no accurate methods of observation can be successfully employed. It is the opinion of the writer that in the endeavor to obtain a large number of fairly accurate data, the quality of the observations is sacrificed for the quantity; for instance, in sounding from a moving steamboat, and also in the small number of soundings taken on a discharge section. The current meter as used by the integration method or run at given depths, checked by flanking, though subject to inaccuracies, is manipulated with greater facility than floats or rods, and unless the latter are used in considerable numbers at short intervals of time, gives better results on the whole. An example of the irregularity of flow of the Mississippi, the change in lateral slope, the obliquity and pulsation of the current, is afforded by observations of the writer at Vicksburg. The river in that locality is approximately as shown in

Mr. Coppée. Fig. 1. During a period of about three years, the writer made careful observations of the depth of water and change in bottom at all stages in the vicinity of the Queen and Crescent Railroad transfer incline at *D* in the figure. These observations disclosed the fact that there was at certain elevations of the water surface, most markedly when the river was at bank-full stage, pulsation in the flow in the neighborhood of this incline. It was first noticed when the water was just over the bank and the only dry spot in the vicinity, consisting of a small mound of cinders dropped from the ash pan of a locomotive, was occupied as an observation station by the instrumentman from which to record the position of the soundings. When first occupied the mound was dry, but in the course of about half an hour the observer was standing in water ankle-deep which receded in about an hour, leaving him on dry ground again. At intervals of about an hour there was a recurrence of the maximum elevation of the water surface, which was found by reading the gauge, located at *B*, to be about 0.3 ft. above the minimum. During the same period the work of dredging the Government canal, shown in the figure, was in progress. At the mouth of the canal, at *A*, a long spar buoy was used by the contractor as an indicator, pointing the direction of the constantly changing currents in the river and serving as a guide to pilots towing scows into the cut. When in slack water the buoy floated in a vertical position; when the cross-current was in the ascendant, an eddy current ran from *B* to *A* and the buoy inclined up stream, the reverse being the case at the culmination of the pulsation.

Fig. 1.

The relative values of apparatus and method, of error of observation and error of deduction, are shown very clearly by the author; but to understand fully the impossibility of obtaining an accurate estimate of the discharge of the Mississippi River one must have been in a light skiff on its surface at about the bank-full stage. Then the difficulty of the undertaking and the absurdity of an attempt to apply directly general formulas, or, indeed, any formulas, to obtain accurate results are fully appreciated. That a difference of 100 000 cu. ft. per second is found between discharge sections seems but natural when

viewing the seething mass of boiling water of this great river at flood stage. Mr. Coppée.

By a careful study of a great mass of data and the construction from it of tables and curves of the elements involved in the evolution of the discharge, the author has established facts that are of the greatest interest and value. The uncertain quantities in the problem vitiate to some extent the value of the deductions, but the results throw light on many mooted questions and give an estimate of future flood heights for a leveed river which will influence the work of protection now in operation and further projects for improvement.

J. B. Johnson, M. Am. Soc. C. E.—While the writer has no desire to criticise the paper in any respect for what it contains, he regrets that the author did not continue his studies a little farther, and give his views on some of the vital questions now confronting the Mississippi River Commission, and through it the United States Congress. If he would not care to express his individual opinion on questions as yet unsettled and in a state of controversy by the members of the Mississippi River Commission, he could properly have given some of the accumulated facts on which approximate estimates might be based. Thus by far the most serious and urgent question pertaining to the discharge of the Mississippi River is the determination of the effect of complete confinement by levees on the maximum discharge throughout the entire length of the river below Cairo, and the probable heights the surface would reach throughout this portion of the river if so confined. He has worked out his conclusions as to these two facts for one particular point upon the river, this being Arkansas City. He has done well to take the flood of 1882 as the standard maximum flood discharge, since it was not only the largest on record, but it was more thoroughly and completely studied and measured than any other great flood has ever been.* It was the writer's fortune to study this flood in the field at its height and to measure the discharge from the main channel through the breaks in the levees from Memphis to the mouth of the Red River. Five parties were measuring the discharge of the water in the main channel at as many different points, and three field parties were studying the discharge over the banks, through the levees, and across the swamps. On his return from this trip, he included in his Mr. Johnson.

* See a paper by the writer on "Great Floods on the Lower Mississippi as illustrated in the Flood of 1882," in the *Journal of the Association of Engineering Societies*, Vol. II, p. 115.

Mr. Johnson. report to the Mississippi River Commission, an approximate estimate of the heights this flood would have reached had it been entirely confined to the river channel.* These estimates were so great as not only to be entirely beyond belief on the part of the other engineers connected with the service, but to produce a general chorus of derision, and the charge that his extraordinary personal experiences on this trip had sapped the foundations of his reason. It was not long, however, before the president of the commission himself became convinced that the effect of flood confinement would be to raise the flood stage for such a volume as passed in 1882 to very much the same heights,† although he has continued to stand alone in that body in this matter.

The popular and almost universal opinion has been that the confinement of the entire flood volume to the river channel would cause it to deepen and widen its bed immediately, and to increase its velocity, so that no appreciable increase in flood height would result. It is interesting, therefore, to find the author estimates that the flood stage at Arkansas City for the passing of a volume like that of 1882 through a channel confined by levees would have been 55.8 ft. As the highest stage at that point in 1882 was 47.1 ft., he estimates an increased stage due to flood confinement at that point of 8.7 ft. The writer's estimate for the increased stage at this point made in 1882 was 9 ft. Since all of his estimates were of course merely approximate, not having the necessary data for a complete study of the problem, he regards this as a practical agreement, and desires to have the author's estimates of the corresponding increased stage at the various other portions of the river for the passing of this same flood throughout a confined channel.

More especially is this desirable at this time, since the Mississippi River Commission has now decided by an almost unanimous vote to shut off the Red-Atchafalaya River system entirely from the Mississippi, thus requiring the Mississippi River channel below the mouth of the Red River to carry the entire flood coming down the channel from above that point. As this lower river has never been able to carry more than about 1 200 000 cu. ft. per second, and would be called upon in the case of this closure, with a flood like that of 1882, to carry about 2 000 000 cu. ft. per second, it is of supreme importance to determine

* See the "Report of the Mississippi River Commission for 1884," Appendix A1.

† See the "Report of the Mississippi River Commission for 1883," Minority Rep. of Gen. C. B. Comstock.

what will be the effect on flood heights of the Mississippi River, in the State of Louisiana, when its maximum volume is thus increased by over 60 per cent. The Atchafalaya River is now a legitimate natural outlet for great floods on the lower river, and if the agents of the United States close this outlet against flood water, and force the main channel to carry the entire flood, which it never has done hitherto, does it not necessarily follow that the United States Government may justly be called upon to pay all damages resulting from the overflow of such levees as now exist on this portion of the river, or first raise these levees to such a height as will give the channel a sufficient capacity to carry these increased volumes? It is said that the Commission has decided to close this outlet, but it is not stated that any action had been taken toward raising the heights of the levees in that portion of the river to enable it to carry safely the greatest floods after this closure has been effected. It is well known that some of the members of the Commission are of the opinion that the effect of closing this high-water outlet would be to cause the main river to discharge the increased volume at the same stage, or possibly a lower one, which has accompanied the discharge of previous floods, reduced by the drawing off through this outlet of several hundred thousand cubic feet per second. This is a question, therefore, of immediate and vital importance, and it would be interesting to have the author's views on the matter. While no one would wish to make a very definite prediction as to the effect of any such cause, it would seem that engineering science ought to be competent, and that the data now accumulated by the Mississippi River Commission ought to be sufficient, to enable an approximate estimate to be made of the results of such a step, and of the increase of flood stage for a confined channel from Cairo to New Orleans. Mr. Johnson.

Arthur Hider, M. Am. Soc. C. E.—In 1892 the writer, who has had supervision of the different discharge parties engaged for several years past in what is known as the Third District of the Mississippi River, including the stations at Arkansas City and Wilson's Point, made some tests of the meters used in discharge work, in order to determine if possible whether the large discrepancies between these two stations, 100 miles apart, were due to errors of observations, unreliability of instruments or other causes. The results of these tests are given in the table on page 324; the discharges show large discrepancies. The discharge at Wilson's Point has been moved forward one day for compari- Mr. Hider.

Mr. Hider. son of results, to allow for the time interval of water passing the upper station reaching the lower.

RESULTS OF HIGH-WATER DISCHARGE OBSERVATIONS AT ARKANSAS CITY AND WILSON'S POINT.*

Date. 1892.	Total discharge at Arkansas City. Cubic feet per second.	Date. 1892.	Total discharge at Wilson's Point. Cubic feet per second.	Difference. Cubic feet per second.
April 22	1 310 000	April 23	1 297 000	13 000
23	1 449 000	25	1 321 000	128 000
25	1 385 009	26	1 344 000	41 000
26	1 525 000	27	1 350 000	175 000
27	1 597 000	28	1 392 000	205 000
28	1 524 000	29	1 406 000	118 000
29	1 546 000	30	1 344 000	202 000
30	1 587 000	May 2	1 378 000	209 000
May 2	1 624 000	3	1 406 000	218 000
3	1 532 000	4	1 279 000	253 000
4	1 733 000	5	1 372 000	361 000
5	1 589 000	6	1 337 000	252 000
6	1 648 000	7	1 382 000	266 000
7	1 613 000	9	1 377 000	236 000
8	1 659 000	10	1 425 000	234 000
10	1 601 000	11	1 327 000	274 000
11	1 493 000	13	1 365 000	128 000
13	1 466 000	14	1 301 000	165 000
14	1 409 000	16	1 268 000	141 000

* See the "Report of the Chief Engineer of the United States Army for 1893," pp. 3686–88.

The current meters used, designated as Price Nos. 38 and 39, were new instruments received from the makers but a short time before the parties took the field. Both of the current meters are of improved construction, and an examination of the plotted ratings, taken at different localities at separate times by both chiefs of parties, shows that they kept their ratings practically uniform during nearly the entire time they were in use.

At the close of the work both meters were found to be in good condition, the rating of No. 39 being unchanged, while that of No. 38 had changed but slightly.

Meter No. 38 was used in the earlier observations at Wilson's Point, and No. 39 at Arkansas City; the later observations at Wilson's Point were made with No. 39, and at Arkansas City with No. 38. The meters were exchanged on May 28th, an old meter designated as No. 5 being used at Arkansas City on May 30th, while waiting for meter No. 38 from Wilson's Point; afterward the observers changed stations in order to find out, if possible, whether the difference in results could be

charged to the instruments, to the difference in the method of doing the field work, or to personal errors of the observers. Mr. Hider.

To account for the large differences shown in this table, the first explanation suggested is to charge the disagreement either to errors in field work or to unreliability in the instruments used. As the chiefs of both parties were skilful, careful and experienced observers, and after changing stations and instruments these discrepancies still existed, some other reasons beside errors of observation were sought to account for the wide differences.

Fluctuations of current will no doubt account for a part of the discrepancy in the two sets of observations, but for probably only a small percentage of the large difference in the earlier observations. An examination of the field notes at both stations shows that there is often a great difference in velocity found on the same meter station from day to day as well as from minute to minute, the velocity sometimes varying as much as 20% on many of the stations in two consecutive minutes. This is probably due to a variety of causes, change in direction of filaments of water due to the effect of winds, or eddies and boils caused by uneven shape of bottom, etc. Whatever the cause, the fact remains that great variations and fluctuations in the current do actually exist.

In order to eliminate as far as possible any error from this cause, it has been customary to occupy each station for five consecutive minutes or even longer so as to obtain a mean velocity for that particular station. To further test the discharge at Arkansas City, a new discharge station was selected at Chicot City about 7 miles above Arkansas City. The discharge was measured with different meters three days in succession and by different observers. These results will be noticed hereafter in detail. At the close of the observations at Arkansas City, an experiment was tried at the Chicot City discharge sections to further test the workings of the meters by placing a meter on each side of the steamboat and measuring the discharge at the same time independently by each meter. Simultaneous observations were taken with both meters, the stations being occupied the usual length of time, five minutes. The distance between the meters on opposite sides of the boat was 25 ft., meter No. 38 being placed on the starboard, and meter No. 39 on the port, side. The results obtained are given in the table on the following page.

Mr. Hider.

Number of Station.	Revolutions in Five Minutes.		Mean Velocity. Feet Per Second.	
	Meter No. 39.	Meter No. 38.	Meter No. 39.	Meter No. 38.
14	116	108	1.59	1.71
13	235	234	3.38	3.34
12	328	314	4.43	4.59
11	337	330	4.64	4.72
10	408	380	5.28	5.67
9	405	383	5.32	5.63
8	499	390	5.47	5.79
7	414	415	5.71	5.75
6	407	354	4.95	5.66
5	402	390	5.41	5.59
4	431	475	5.49	5.98
3	479	369	5.14	5.56
2	344	371	4.37	4.81
1	251	235	3.40	3.55
	5 056	4 747	64.58	68.35

The discharge given by meter No. 38 is 1 108 256, and by meter No. 39 is 1 035 156, cu. ft. per second, meter No. 38 giving a discharge 7.06% greater than meter No. 39. The mean of the velocity determined by dividing the sum of the revolutions by the time of the observation in seconds and applying the ratings determined for each meter gives as the mean velocity on the starboard side for meter No. 38, 4 882 ft. per second, and that for meter No. 39 on the port side, 4.613 ft. Meter No. 38 gave a velocity 5.83% greater than meter No. 39 on the opposite side of the boat. It will be noticed that the results obtained by meter No. 38 are always somewhat in excess of those obtained by meter No. 39. This, if the velocity was the same on both sides of the boat, is contrary to what might be expected from stations 14 to 7, as the velocity is continually increasing to that point, so that meter No. 38 on the starboard side should really give a somewhat less velocity than meter No. 39 on the port side; from stations 1 to 7 the velocities decrease, and meter No. 38 should give greater velocities, which it does.

These observations at Chicot City were taken July 2d, when the river was at 44.85 ft. on the Arkansas City gauge, or 5 ft. below high water. The river was falling at the rate of 0.5 ft. in 24 hours, which at this stage is a rapid fall and would tend to produce a considerable shifting and a consequent variability in the current at any one point in the section. This is shown by an examination of the field notes.

Other observations were made August 17th, 1892, by the writer

at Greenville, Miss., after the discharge measurements had been concluded, to further test the workings of these two meters. Mr. Hider.

The meters were placed as before, one on each side of the steamboat, 25 ft. apart. Four stations were occupied for five minutes each, and then the meters changed to opposite sides of the boat, and the observations repeated.

The results are given in the following table:

Meter No. 39 on Starboard Side.		Meter No. 38 on Port Side.	Meter No. 38 on Starboard Side.		Meter No. 39 on Port Side.
	Revolutions.	Revolutions.		Revolutions.	Revolutions.
Station 1.....	26	30.5	Station 1a....	31.5	30
	30	37		26.5	29
	34.5	39		34	35
	33.5	34		30.5	36
	36.5	36		27.5	30
Mean	32.10	35.3		30.0	32.
Mean velocity..	2.370	2.405		2.215	2.285
Station 2.....	37	37	Station 2a....	34	36
	36	32		33	31
	34	35		—	—
	34	39		34	35
	34.5	38		35.5	34
Mean	35.3	36.2		34.125	34.00
Mean velocity..	2.422	2.433		2.360	2.356
Station 3	45	46	Station 3a....	45	46
	49	50		45	45
	48.5	49		48	43
	53.5	53		50	49
				56	54
Mean	48.75	49.5		48.8	47.4
Mean velocity..	2.904	2.865		2.843	2.826
Station 4.....	47	50	Station 4a....	38	37
	46	46		38	35
	44	45		39	36
	41	43		37	43
	41	41		44	40
Mean	43.80	45.00		39.20	38.1
Mean velocity..	2.710	2.725		2.553	2.510

A comparison of the means of these observations shows a remarkably close agreement, and demonstrates the reliability of the instruments and the correctness of the ratings used in the discharge observations.

Mr. Hider.

MEANS.

Station.	Meter No. 39 on Starboard Side.	Meter No. 38 on Port Side.	Station.	Meter No. 38 on Starboard Side.	Meter No. 39 on Port Side.
	Velocity per second.	Velocity per second.		Velocity per second	Velocity per second.
1.....	2.370	2.405	1*a* ...	2.215	2 285
2.....	2.422	2.433	2*a* ...	2.360	2.356
3.....	2.904	2.865	3*a* ...	2.843	2 826
4.....	2.710	2.725	4*a* ...	2.533	2.520
Mean.	2.606	2.607	....	2.488	2.488

At the time these observations were taken all conditions were extremely favorable for exact work, no wind, river calm, Greenville gauge 14.5 ft., meter submerged 15 ft., and river nearly stationary. The interval between observations at Stations 1 and 1*a* was about 40 minutes; 2 and 2*a*, 30 minutes; 3 and 3*a*, 20 minutes; 4 and 4*a*, 10 minutes, the boat crossing the river from Station 1 to 4, and, returning, taking the stations in the reverse order. It will be noticed that during this time the velocity on some of the stations varied considerably during the interval between observations.

Another test was made the same day by placing both meters on the same rod, one above the other, and 2 ft. apart; a 60-lb. lead was attached to the bottom of the rod 18 ins. below the lower meter. The

	Meter No. 38 Above.	Meter No. 39 Below.	Meter No. 39 Above.	Meter No. 38 Below.
	Revolutions.	Revolutions.	Revolutions.	Revolutions.
Station 1.....	28	30	31.5	31
	31	33	31.5	31
	33	34	29.5	30
	33	36	31	32
	32.5	34	29.5	30
Mean....	31.5	33.4	30.3	30.4
Mean velocity....	2.265	2.333	2.220	2.235
Station 2.....	36.5	38	35	35
	39	40	38	38
	35	34	40	40
	33	33	41	44
	35	34	37.5	38
Mean.....	35.3	35.8	38.3	39.0
Mean velocity....	2.404	2.420	2.500	2.538

lower meter was submerged 15 ft., two stations were occupied, and observations at each station lasted five minutes. A copy of the notes is given in the table at bottom of preceding page. Mr. Hider.

It will be observed that the lower meter gave slightly higher velocity. In one case the mean gave an increase of 0.042, and in the other case of 0.027 ft. per second, caused probably by the disturbance in the current due to the obstruction formed by the weight 18 ins. below the lower meter.

A still further test of the working of these two meters was made the same day. The steamboat, with a meter submerged 15 ft. on each side, was run slowly across the river, keeping on range ; the revolutions of each of the two meters were counted and recorded each minute. Meter No. 38 was on the starboard side and No. 39 on the port side. The results of the observations were as follows :

No. of Minutes.	Meter No. 39.	Meter No. 38.	No. of Minutes.	Meter No. 39.	Meter No. 38.
1	33	36	15	59	60
2	44½	44	16	67	70
3	53	54	17	54	51
4	58	58	18	52	52
5	64	66	19	49	45
6	66	66	20	41	41
7	68	68	21	32	36
8	69	69			
9	69	69			
10	69	69	Total revolutions	1 196½	1 211
11	68	69	Revolutions per second	.950	.961
12	64	69	Mean velocity	4.055	4.050
13	62	60			
14	59	60			

These observations again show a very close agreement, the difference in mean velocity after 21 minutes of consecutive observations being only 0.1 per cent. The three sets of observations, which were made with extreme care and under very favorable conditions, prove clearly the reliability of the meters used in the discharge observations, and also the correctness of the ratings used in the computations, which were the same as those used in these experiments.

An examination of these results shows that the velocity at any one point in the sections varies to a considerable extent in a short period of time. This being admitted, the variations in the discharge, comparing one day with the next, cannot be fairly charged to irregularity of the meter's action, but much more probably the difference is due to the variability and fluctuations in the velocity of the current itself.

Mr. Hider. There is no reason why an instrument as carefully constructed as one of these current meters should not register the facts, and there is no doubt but that the current meter does register the velocity that actually exists at the time the station is occupied.

Another test of the accuracy of the meter work is given by the result of double-float observations. The lower float was submerged six-tenths of the depth, the same as for meter work, and the observations were taken at both stations; the discharges were also measured by the meters the same day. The results are given in the following table:

Place.	Date.	Cubic Feet per Second.		Percentage of difference between meter and floats.
		Floats.	Meter.	
Arkansas City.....	May 14....	1 407 489	1 395 708	— 0.08
"	" 23....	1 486 551	1 436 379	— 3.37
"	June 20....	1 403 133	1 379 797	— 1.66
"	" 30 ...	1 198 481	1 138 237	— 2.21
"	July 1....	1 182 964	1 138 237	— 3.78
Wilson's Point.....	May 19....	1 245 815	1 273 454	+ 2.22
"	" 27....	1 297 915	1 345 600	+ 3.68
"	" 30....	1 282 161	1 257 200	— 1.94
"	June 6....	1 227 766	1 213 740	— 1.14
Chicot City.....	June 30..	Meter, morning. 1 047 756	Meter, afternoon 1 066 256	+ 1.73
"	July 1....	Floats. 1 064 356	Meter. 1 022 056	— 3.97

The preceding table shows as close agreement with meter work as could be expected, when it is considered that the meter was run in the forenoon, the section sounded and the floats run in the afternoon, some three or four hours intervening between the two sets of observations, the same soundings for determining the area of the section being used to obtain the final discharge with the meter and double floats.

The test at the Chicot City station, 7 miles above Arkansas City, made at the close of the discharge observations, by placing a meter on each side of the boat, shows a wide discrepancy when the discharges at the two stations are compared.

Date.	Arkansas City.		Chicot City.		Difference.
	Meter.	Discharge per second.	Meter.	Discharge per second.	
		Cubic feet.		Cubic feet.	Per cent.
June 28.....	39	1 188 987	38	1 281 656	— 7.79
" 29.....	39	1 179 450	38	1 250 156	— 6.00
" 30.....	38	1 174 778	39	1 047 756	+10.81
July 1.....	38	1 140 517	39	1 022 056	+16.19

Double floats were run at Arkansas City on June 30th. Meter gave 1.73% less than floats. Mr. Hider.

Double floats were run at Arkansas City on July 1st. Meter gave 3.78% less than floats.

Double floats were run at Chicot City on June 30th. Meter gave 1.73% less than floats.

Double floats were run at Chicot City on July 1st. Meter gave 3.94% more than floats.

It would seem that the Arkansas City discharge should be somewhat greater than the discharge of Chicot City during this time, due to the draining of the reservoir between the two stations, as the river was falling at the rate of 0.5 ft. per day during this time.

The field notes of velocities obtained on these four days are given in the following tables:

ARKANSAS CITY.

Station.	Meter No. 39.		Meter No. 38,	
	June 28th.	June 29th.	June 30th.	July 1st.
1	3.274	3.041	3.07	3.00
2	5.228	5.011	5.04	5.12
3	5.622	6.025	5.97	5.35
4	6.142	6.297	6.77	6.23
5	5.954	5.815	6.00	5.78
6	5.599	5.599	5.69	5.30
7	4.923	5.360	5.52	5.36
8	5.054	4.882	5.00	4.98
9	4 337	4.290	4.29	4.23
10	3.944	3.859	3.99	3.96
11	5.691	3.799	3 76	3.55
12	3.274	3.127	3.48	3.07

CHICOT CITY.

Station.	Meter No. 38.		Meter No. 39.		
	June 28th.	June 29th.	June 30th. A. M.	June 30th. P. M.	July 1st.
14	1.97	1.64	1.831	1.927	1 592
13	3.32	3 30	3.111	3.319	2.809
12	4.81	4.54	4.547	4.597	4.416
11	5.91	5.28	4.547	4.880	4.352
10	6.68	6.33	5.062	4.852	4.290
9	6.25	5.82	5.367	4.992	5.463
8	6.09	6.02	5.282	5.437	5.151
7	6.27	6.22	5.166	5.527	5.198
6	6.13	5.87	5.162	4.806	5.139
5	6.44	6.01	5.026	4.992	5.216
4	6.16	6.47	5.255	5.282	5.151
3	6.29	5.97	5.127	5.437	5.023
2	5.34	5.27	4.482	5.139	4.509
1	3.80	3.74	3.551	.....	3.418

Mr. Hider. These observations show that the locus of the maximum velocity at the Arkansas City station during the four days was constant; at Station No. 4, at Chicot City, it moved from June 28th to June 29th, from Station No. 10 to Station No. 4, 1 800 ft.; on June 30th it was at the same point. From the morning to the afternoon of that day it moved from Station No. 4 to Station No. 7, 900 ft., and from the afternoon of June 30th to the morning of July 1st it moved from Station No. 9 to Station No. 7, 600 ft. This shifting of the current and the consequent variation of velocity while taking meter or float observations will account for many seeming discrepancies in the results, for the reason that it takes from three to four hours to make a set of either of these measurements, and during this interval considerable variation in velocity at some of the stations may exist, due to the meter measurements following in the wake of the movement of the point of maximum velocity, or *vice versa*.

That the change in the velocity curve due to the shifting of the currents caused by winds, caving banks, change of direction, etc., and consequent change in local slope is at times a considerable source of error in the results obtained by present methods is shown by an examination of the field notes. Taking the discharge at Chicot City as an example, as the meter was run beginning with Station No. 14 and occupying the stations in regular order until the other side of the river was reached, it is evident that on June 28th and 29th the measurements would follow the movement of maximum velocity, and therefore give results somewhat too large. On June 30th and July 1st the reverse would be the case, the results being too small. As the locus of the maximum velocity on the Arkansas City station remained fixed, the results there would probably be more accurate.

The meter and float observations both show that on June 28th and 29th the Chicot City observations were larger than the Arkansas City observations, and on June 30th and July 1st much smaller.

These meter results are checked by float observations which agree very well. As the meters were changed as well as the observers, and the work checked by the float method, there is no reason to doubt the reliability of the results as given by the instruments, or that they were the actual velocities at the time. It seems evident that the change in locus of maximum velocity and mean velocity due to change in local slope is the cause of a part of the difference in the measurements.

The change in reservoir capacity between the two stations would also have a slight effect, and the form and depth of section would have some influence, but neither of these causes nor all combined are sufficient to account for the discrepancies above referred to. There is probably a considerable loss of water between two stations 100 miles apart, from infiltration through the bottom and sides of the river, which is from 60 to 100 ft. deep and flows through an alluvial bed, due to the filling up of an immense reservoir drained during the preceding low water season: how much this may be is purely a matter of conjecture. Mr. Hider.

The wide difference between the computed discharges at Arkansas City and Wilson's Point in 1890–91 and 1892 suggested to the writer a possible improvement in the method of measuring the discharge, which is referred to in the paper as the flanking method. This change in method was recommended to C. McD. Townsend, M. Am. Soc. C. E., the engineer officer in charge of the district at the time these tests were made, and, in connection with the former or stationary method, has been in use since. The usual method was to divide the entire width of the river into sections of about 300 ft., ascertaining the velocity in the middle of the section by running a meter for five consecutive minutes, and assuming the average velocity so obtained as the mean velocity of the entire 300 ft.

The change in method may be briefly outlined as follows:

For the Stationary Method.—To divide the cross-section of the river into a greater number of equal parts, twenty-four instead of twelve.

For the Flanking Method.—First, to submerge the meter 15 ft., or if necessary 20 ft., below the surface, so as to be below any disturbance caused by the movement of the boat; second, to start the boat at a slow rate of speed across the river, keeping on the discharge range, noting the registration of the meter, and the time of passage between each division; third, to sound the section during or immediately after the meter observations are taken.

The advantages of these changes are the following:

In the stationary method the accuracy of the work is increased by doubling the number of divisions into which the section of the river is divided, and obtaining a mean velocity of each division.

In the flanking method, the meter can be more easily manipulated at a depth of 15 to 20 ft. than 40 or 50 ft. below the surface.

Mr. Hider. Three or four discharges can be measured the same day if necessary.

By noting the time and registrations between each division and making the proper correction for lateral motion of the boat and depth of submersion of the meter, a nearer approximation to the true results will be obtained than by the usual method of assuming that the average velocity taken in the middle of a 300-ft. section in the river will give a mean velocity for the entire 300 ft., as the field notes show that the current is extremely variable and continually shifting. Another advantage is that the usual time of taking the meter observations by the stationary method is from two and a half to three and a half hours, while by the flanking method a complete set of observations across the river and back does not occupy more than one-third of the time, allowing two sets of measurements to be made the same day.

The true velocity can be readily deduced by the simple formula $v = \sqrt{v' - v''}$, where v is the true velocity, v' is the velocity in feet per second as given by the meter between stations, and v'' is the velocity of movement of the boat across the range between stations.

By the stationary method it is assumed that the average velocity of a filament of water from 900 to 1 800 ft. in length passing through a section of the river 150 to 300 ft. wide represents the mean velocity of the water flowing through that section.

By the flanking method there is obtained an integration of all the different velocities of the filaments of water flowing through the section at a uniform depth below the surface. Applying the proper correction for the motion of the boat and for depth of immersion should theoretically give more accurate results than the stationary method.

Assuming that the velocities and directions of flow through the section during the time interval, generally a little less than an hour, remain constant while the observation is being taken from one side of the river and back, in crossing the river in the same direction as the lateral flow, the meter, after the proper correction is made for the motion of the boat, would register the normal flow through the section, less the decrease of discharge due to the lateral movement of the water. Returning across the river against the direction of the lateral flow, the meter would register the normal flow plus the increment due to the lateral movement of the water. It is immaterial whether all the filaments of the water are inclined in the same direction or not, it

Mr. Hider.

DISCHARGES AS DETERMINED BY STATIONARY AND FLANKING METHODS.

Location.	Date.	Gauge.	Discharge, Flanking Method.			Discharge, ordinary method.	Remarks.
			R. to L. Bank.	L. to R. Bank.	Mean.		
Arkansas City	July 8th, 1895	13.5	295 379	270 890	283 135	.....	River raising rapidly.
"	" "	13.9	.....	.....	.....	302 493	
"	Aug. 12th, 1895	18.3	371 730	333 125	352 428	.....	River falling rapidly.
"	" "	17.9	.....	.....	.....	353 531	
"	Oct. 12th, 1895	— 1.2	130 795	74 383	102 589	.....	River falling slowly.
"	" "	— 1.24	.....	.....	.....	120 911	
Wilson's Point	July 24th, 1895	13.9	379 531	367 266	373 399	.....	River falling rapidly.
"	" "	13.8	.....	.....	.....	370 011	
"	Oct. 21st, 1895	— 2.3	104 091	91 717	97 906	.....	River falling slowly.
"	" "	— 2.3	.....	.....	.....	102 054	
Warrenton	Aug. 2d, 1895	13.86	274 302	356 105	315 204	.....	River stationary.
"	" 3d, 1895	13.86	.....	.....	.....	361 641	
"	Oct. 26th, 1895	— 3.73	85 959	108 103	97 031	.....	River falling very slowly.
"	" 26th, 1895	— 3.76	.....	.....	.....	101 657	
"	" 28th, 1895	— 3.90	86 967	109 663	98 315	.....	River falling very slowly.

Mr. Hider. is seldom that the direction of the current in reference to the discharge range is relatively the same on both sides of the river during high water.

The normal discharge at right angles to the section would be the mean of the measurements taken across the river and back again, and the excess of lateral flow in one direction one-half their difference.

A few observations recently taken by both the stationary and flanking methods at stations about 100 miles apart when the disturbing elements, boils, eddies, irregularity of flow, etc., to which the river is

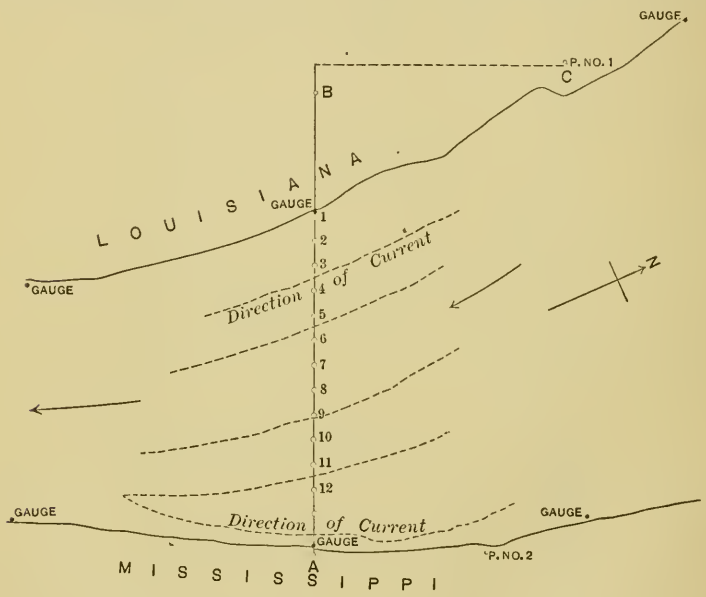

FIG. 2.

subject at high water were at their minimum, are given in the table on the preceding page.

The reason of the wide discrepancies in results between the two methods at the Warrenton station can be seen by an inspection of Fig. 2, showing the direction of the current relative to the discharge range. The importance of taking this lateral flow into consideration in all discharge measurements is evident.

Mr. Townsend. C. McD. TOWNSEND, M. Am. Soc. C. E.—The author's deductions as to the heights to which the Mississippi River may rise when confined are full of vital importance to those interested in levee matters,

but it is believed he has rendered a still more valuable service to the engineering profession as a whole by gracefully removing the mask from that venerable formula $v = c\sqrt{rs}$. When it is shown that the flow of water in rivers fulfils none of the conditions on which the formula is based, the cross-section not being uniform, the velocity constant nor the filaments of water flowing in parallel paths, it is not surprising that difficulty is experienced in its application. The author, however, makes a much more serious attack when he demonstrates that on any section the velocity may not bear a direct relation to the hydraulic mean depth, but an inverse relation, and that the local slopes may not bear the relation to v that the formula indicates. Mathematics is but a process of reasoning, and, starting with false premises, cannot lead to proper conclusions. The author appears to think there is an opportunity to convert falsehood into truth by multiplying by a variable, and the question arises as to what is the value of the quantity that will convert the square root of the product of the hydraulic mean depth of an accidental section and an imaginary slope called a mean slope into another imaginary quantity called mean velocity. It is shown that the location and form of the discharge section bear an important relation to this value. An inspection of the formula shows that c can be made to vary between zero and infinity on most rivers by varying the location of the discharge section, selecting the edge of a fall in one case near the river's source, and in the other a section near the mouth under tidal influence when the slope happens to be negative. The value of c depends on the relation that the discharge section bears to a mean river section and local slopes to mean slopes; in other words, on the fallacies that enter the original premises. The necessity for the author's caution in applying the formula is therefore evident. Every variety of falsehood changes the value of c, and it therefore results that c varies with the section, with the stage, with the slope, and with various accidental conditions. When the author constructs an imaginary river where $v = c\sqrt{rs}$ represents a true relation, the value of c becomes determinate.

Mr. Townsend

An apt illustration of the above thought is given in the paper.* The measurements of discharge in 1893 at Bachelor's Bend and Warfield's Point were taken to determine what influence, if any, the location of the discharge section has on discharge measurements. The

* See *Transactions*, Vol. xxxiv, p. 401.

Mr. Townsend. section at Bachelor's Bend was located in a sharp curve, and the section at Warfield's Point on the crossing below it; the large error found at the former locality simply indicates that the section selected differs widely from the mean river section, and the local slopes from mean slopes, proving that the section was a good one for the purpose intended, but very poor for obtaining correct discharges.

Text books offer excellent advice to select a location for a discharge section in a straight reach of uniform cross-section where the filaments of water are flowing in parallel lines. If this advice could be followed, not only could accurate discharges be obtained, but the author's troubles with the formula $v = c\sqrt{rs}$ would disappear. The whole trouble arises becauses the Mississippi is not that variety of a river; straight reaches are few, uniformity of cross-section is an iridescent dream, the threads of flow are most complicated skeins. In obtaining high-water discharges, it is essential to select a location where the flow of water is unobstructed by trees and undergrowth, as the determination of discharges under such conditions cannot be accurate. In the upper portions of the river this limits the discharge stations to a few localities. The discharge must, therefore, be measured where opportunities occur, not where theoretical considerations would indicate it was most desirable, and to the necessarily faulty location of the discharge section the writer attributes many of the observed discrepancies in discharge measurements. The fact that there was a deflection in the threads of current at Arkansas City of 10° 30′ was not the controlling feature in the process of reasoning given by the author,* but that the floats followed paths which were approximately arcs of circles concave to the Mississippi bank, with radii of curvature of between 6 000 and 7 000 ft., that there was a marked difference in the water level on the two banks of the river, and that the local slopes on the Arkansas side were very gentle, and on the Mississippi side very steep.

If the tendency of water under the action of a centrifugal force is to move in spiral paths, the conditions at Arkansas City are favorable to the development of such action. At Wilson's Point, on the contrary, though some of the filaments of water follow curved paths, some are concave to the Mississippi shore and some to the Louisiana shore, a condition not favorable to the development of centrifugal force.

A direction-current meter will not solve the difficulties which arise in

* See *Transactions*, Vol. xxxiv, p. 373.

such a case. It may give the direction and intensity of a resultant force, but neither the intensity nor direction of the components into which it must be divided will be known. In a pipe or canal the determination of the cross-section is simple, but it is far from being the case in a river. It has been customary to select a section approximately at right angles to the direction of the current, and if the filaments of water move in right lines, the projection of those lines on planes perpendicular to the discharge section gives a correction to the velocity that enables discharge measurements to be made accurately, but if the spiral motion of the water be admitted, then not only is the resultant velocity greater than the true velocity with which the water is running down the river, but the area of the section selected varies from that of the proper section by an amount dependent upon the inclination of the spiral. The multiplication of these two quantities together may lead to large errors. This fact, however, does not materially affect the author's deductions. It is possible that an unknown quantity x may be a better expression than cubic feet per second for the unit, on which his numerical relations depend. The same discharge curve, however, will be obtained if x's of the same value are compared. The analysis to which the discharge measurements are subjected tends to produce such a result. It may be that the discharge observations rejected at Wilson's Point as too small may give the most accurate discharge in cubic feet per second, but they are properly rejected when an investigation indicates that they are not based on the same x that is the unit for the Arkansas City discharges. Mr. Townsend.

The author's deductions of the height that the confined river will attain reopens a subject which has been discussed in the Mississippi valley since the days of Humphreys and Abbott. Two schools of engineers have arisen, one, of which the author by the publication of his paper becomes an able exponent, maintaining that the height that floods will ultimately attain in a leveed river can be mathematically determined; the other, that the influence of the confined river in enlarging the river channel will vitiate any mathematical deduction. With one school, the author's arguments will appear conclusive; with the other, the essential item will be found in the statement* that "the section at Arkansas City, with all its temporary mutations, remains the same."

*See *Transactions*, Vol. xxxiv, p. 489.

Mr. Townsend. The levees have now been constructed to such dimensions that if they are capable of producing any effect on the river channel, there should be evidences of the fact, and it is the writer's purpose to sum up the evidence that exists of the enlargement of cross-section that has accompanied levee construction, and to discuss the effects which enlargement of section will produce on flood heights. This discussion leads to a deduction of levee grades by a process of reasoning entirely independent of that pursued in the paper. For those engineers who have not been heretofore interested in levee construction to understand the discussion some information is necessary concerning the work which has been done on the Mississippi River in recent years. It is proposed, therefore, to preface the discussion by a description as brief as practicable of the levees of the Mississippi River.

TABLE A.—CUBIC YARDS IN LEVEES.

DISTRICT.	LENGTH IN MILES.		CUBIC YARDS IN LEVEES.	
	River.	Levee line.	1882.	1895.
Lower St. Francis	223	118	.....	3 637 083
Upper White	34	32	.....	2 103 132
Lower White	45	33.5	.....	2 108 710
Upper Yazoo	120	123.9	2 600 000	10 434 395
Lower Yazoo	210	190	5 968 726	19 700 000
Upper Tensas	118	*90.8	1 788 300	9 340 917
Middle Tensas	88	87.4	3 181 000	9 357 861
Lower Tensas	155	121.6	5 289 000	10 906 816
Atchafalaya	123	*104	5 043 784	12 295 714
La Fourche	71	*71.3	2 735 284	5 592 863
Barataria	58	60	501 860	1 736 870
Pontchartrain	123	125.8	4 264 638	9 637 297
Lake Borgne	58	60	1 367 680	2 096 549

* Length on Mississippi River.

Prior to the war between the States, the fronts of the different basins were covered by lines of levees, though of weak profile and insufficient height. During the war the contending armies frequently cut the lines, and the floods of 1862 and 1865 very generally destroyed what the troops left. Spasmodic efforts were afterward made by local authorities to repair their levees, but the flood of 1882 again devastated the entire valley. At this period the United States Government offered assistance in reconstructing the levee line, and since that date the work has been vigorously prosecuted by the general government and by State and local authorities. In Table A are given the cubic yards in the levee lines of the various districts after the

flood of 1882, and on June 30th, 1895, and the total length by river of the different districts. Fig. 3 shows cross-sections of levees such as existed in the Upper Tensas District, in which Arkansas City is situated, in 1882, and such as will be found throughout the district before the flood of 1896, when the work now under contract will be completed. The section of 1882 has been determined by dividing the contents of the levee line that then existed by its length. The broken and dotted line of the diagram shows a section to restrain a flood of the dimensions indicated in the paper, with an allowance of 1½ ft. for wave-action. It may be noticed from this diagram that the forms adopted for the levee section differ materially from those which obtain in reservoir construction. The causes of this difference arise from the impracticability of finding in an alluvial valley the foundation impervious to water that is an essential condition in reservoir construction. The material for constructing the puddled core is also not readily obtainable, and its value is not so evident when it can be readily flanked by water passing through a permeable foundation into the embankment beyond. Mr. Townsend.

The essential condition of levee construction is a mound of earth of sufficient height so that water cannot flow over its top; of sufficient mass, so that surfaces of saturation cannot be formed through it on which the superincumbent material will slide; of sufficient width of base, so that the water flowing through the permeable foundation will not have sufficient force to remove any material, it being assumed that the earth has been placed with care, so that there are no channels left across the embankment through which the water can directly flow. This mound of earth must be protected from the eroding action of rains, and its river surface from erosion by river currents and waves during floods. The grade must be somewhat higher than the highest flood to afford protection against wave action during storms.

With the extreme variations in permeability which are found in the soils of which levees are constructed, variations in form become necessary, but practical experience with the soils of the Mississippi bottoms has developed certain forms which are of general application. A slope of 1 on 3, when well sodded with Bermuda grass, has been found to resist the action of rain, of river currents and of minor waves. In localities exposed to the full force of waves through long reaches of

Mr. Townsend. the river, it is necessary either to reinforce the sod with some more permanent form of revetment, or to adopt a gentler slope.

A width of crown varying from 6 to 10 ft. generally exists. The width to be given to the crown is interdependent with the slope to be given the land side of the levee. An 8-ft. crown affords convenient space for patrolling the levee line during floods, and storing such material as may be required to repair any damage which may be inflicted by storms. With a width of crown of 8 ft., a land slope of 1 on 2 can be safely employed until the levee attains a height of 6 ft.; with higher levees, unless the soil is especially adapted to levee construction, there is a tendency to form surfaces of saturation through the mass on which the upper portion is liable to slide. Levees not exceeding 12 or 14 ft. require a land slope of 1 on 3; above these

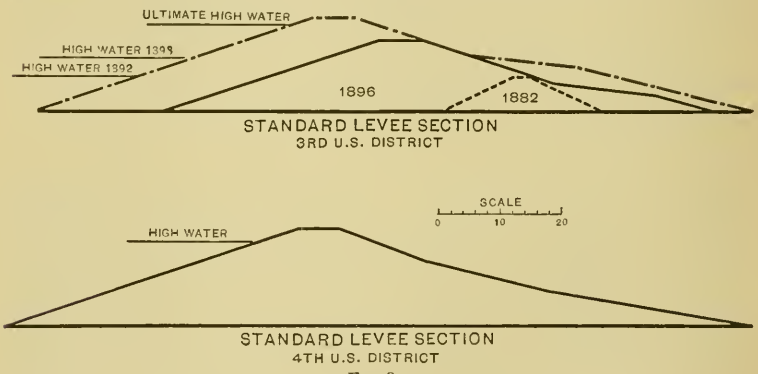

FIG. 3.

heights in the upper basins of the river, trouble begins to be experienced with the foundation, and still gentler slopes are necessary, or the same results may be attained by adding to the base a mound of earth termed a banquette. These banquettes have been built of various dimensions and slopes, varying from 10 to 40 ft. in width, their crests from 6 to 10 ft. from crown of levee, with land slopes from 1 on 3 to 1 on 5, and with top slopes from 1 on 40 to 1 on 10.

In the Fourth United States District, recognizing these principles, a form of section shown in Fig. 3 has been employed. The width of crown is 8 ft., river slope 1 on 3, the land slope for a vertical distance of 6 ft. from the crown is 1 on $2\frac{1}{2}$, for the next 6 ft., 1 on 4; below this, 1 on 6. The objection to this form is that it does not lend itself readily to levee enlargement. Owing to lack

of funds in nearly all the districts it has been impracticable to build levees of the dimensions it is recognized they must ultimately attain. The hope is entertained that several years will elapse before a great flood will occur, and safety is sought against the minor rises which are sure to happen. A weak section is first adopted, which is afterward enlarged as it exhibits signs of failing, and more money becomes available. A trapezoid with the width of the crown adjusted to the financial conditions of districts, and slopes of 1 on 3, is well adapted to this method of treatment. If its base shows signs of failing, it can be strengthened by a banquette even during a flood, and it can be readily enlarged on the river side without destroying the sod on the other slope. The section shown in Fig. 3 must be enlarged on the land side or the slopes readjusted. While these levee sections have resulted from practical experience rather than from theoretical considerations, it may be noted that the sections most generally employed in recent years insure an angle of repose of dry earth on wet clay of 14° when water is at the top of the levee, the limiting angle for such materials. When compared with European practice, it will be found that European levees have in general a greater width of crown and steeper land slopes than are found in Mississippi levees, but about the same width of base.* Mr. Townsend

The engineers engaged in levee construction have heretofore adopted what has aptly been termed provisional grades for their levees. The impossibility of restraining great floods with the money available was early recognized in the various districts, and grades were adjusted to their financial requirements. In 1882 the gaps in the levee line were closed to the height of the flood of 1882. As more money became available, levees were built to a grade 3 ft. above the flood of 1882. In recent years the following provisional grades have been adopted: In the Third United States District and by Louisiana authorities, 3 ft. above the highest flood; in the Fourth United States District, $2\frac{1}{2}$ ft. above the highest flood, and in the Upper Yazoo District, 4 ft. above the flood of 1890; in the Lower Yazoo District, 4 ft. above the flood of 1891; in the St. Francis District, 3 ft. above the flood of 1882. The grades adopted by the Lower Yazoo and Third United States Districts are nearly equal to the United States grade, being a little higher than the local grade at the upper end of the district and a little lower at the lower

*See paper entitled, "Some Notes on the Holland Dikes," *Transactions*, Vol. xxvi, p. 559.

Mr. Townsend. end. They are both about 1 ft. below the grade of the Upper Yazoo District.

By March 1st, 1896, it is anticipated that the levees constructed in these districts will be generally brought to standard section and to provisional grade, and in a number of districts higher grades than those specified have recently been adopted.

While the levees are in an incomplete condition their influence on the high-water discharge of the Mississippi River is already perceptible. In the following table are given the maximum measured discharges in the river channel, and the estimated discharge across the alluvial valley at the latitude of Lake Providence in 1882 and from 1890 to 1893:

Date.	Measured discharge.	Estimated flow across the valley.
1882	1 057 000	2 000 000
1890	1 288 000	1 720 000
1891	1 346 000	1 400 000
1892	1 430 000	1 760 000
1893	1 498 000	1 750 000

This table is derived from the reports of the Mississippi River Commission, and has not been submitted to the rigid analysis that characterizes the paper. The results are, however, given as reported, as they have been used as arguments for solving the question of ultimate flood heights. The discharges at the latitude of Lake Providence are the roughest approximations, as explained in the paper. The increased river discharge has been accompanied at Lake Providence by an increase of 3½ ft. in flood heights. In Table B are given the flood heights since 1872 at the principal gauge stations on the river from Helena to the mouth of the Red River.

While no great flood has been entirely restrained, the gradual increase in the area of land protected from overflow is as marked as the increase in flood heights. During the floods of 1882, 1883 and 1884 the valley of the Mississippi River was generally overflowed, but in 1890 between 80 and 85% of the Yazoo Basin was protected. In 1891 the amount of land overflowed in this basin was still less, and in 1892 and 1893 the levees afforded it complete protection. The Arkansas districts have been generally overflowed during every flood, though in the Upper Tensas District the levee line was successfully maintained in 1891.

Below the Red River in 1890, on the right bank, a continuous line of 180 miles was maintained, and on the left bank there were 200 miles with but one break. In 1891, two crevasses occurred—one, the Ames crevasse, overflowed a territory of about 2 000 square miles; the second, the Fernsley crevasse, occurred as the river was falling; it attained a width of but 97 ft., was readily closed and inflicted only local injury. In 1892 there was but one crevasse in the Mississippi levees of the Atchafalaya Basin. In 1893 no crevasses occurred in the Atchafalaya nor in the La Fourche Basin above New Orleans. Mr. Townsend.

A summary of the evidence of enlargement of section accompanying levee construction will now be given. If the discharge observations

TABLE B.—Flood Heights Since 1872.

Year.	Cairo.	Helena.	White River.	Arkansas City.	Lake Providence.	Vicksburg.	Natchez.	Red River.
1872....	39.2	39.0	40.2	....	35.1	39.5	39.8	39 4
1873....	41.5	40.0	42.9	....	36.1	40.6	40.1	39.0
1874....	47.4	45.8	....	....	37.4	45.7	45.6	47.0
1875....	43.7	42.4	45.0	....	37.3	43.0	41.8	40 4
1876....	46.4	44.8	46.7	....	37.9	44.9	43.8	45.4
1877....	40.5	41.8	44.6	....	35.8	41.6	40.7	40.5
1878....	37.0	38.7	....	....	35.8	40.9	39.2	....
1879....	36.5	37.2	....	....	36.0	39.4	36.8	35.9
1880....	44.6	43.7	46.5	45.1	38.0	43.1	43.5	44.0
1881....	45.8	43.7	45.7	44.3	36.2	41.8	40.8	40.1
1882....	51.9	47.2	48.4	47.1	38.3	48.7	47.7	48.5
1883....	52.2	46.9	48.0	46.3	36.5	43.8	44.0	45 2
1884....	51.8	47.0	47.9	46.5	38.4	49	47.4	47.3
1885....	39	40.7	43.6	42.6	35.5	42.4	42.6	41.9
1886....	51	48.1	48.2	46.9	37.9	44.1	43.7	41.9
1887....	48.5	46.4	47.7	46.8	38	44.7	44.2	43
1888....	45.4	42 8	45.5	45.4	38.1	44.2	43.4	41.7
1889....	35 4	34.1	37.6	36.2	29.4	34.4	34.1	34
1890....	48.8	47.7	50 4	49.5	41	49	48.6	48.6
1891....	46.2	44.7	47 7	48.2	41	48.3	46.5	45.5
1892....	48.2	45.7	49.3	50	41.9	48.4	48	48.8
1893....	49.3	47.9	49.5	50.3	41 8	48.3	46.8	47.7

quoted by the author are examined, it will be seen that while the discharge section at Arkansas City in 1893 has about the same area as in 1884–85, there was an increase in area from 1890 to 1893 which compensates for a reduction in area from 1885 to 1890. During 1890, 1891, 1892 and 1893 great floods occurred, also in 1882, 1883 and 1884, while from 1885 to 1889 the waters of the river were relatively lower, notably in 1889. Colonel Suter has informed the writer that from surveys made near Arkansas City in 1879 at about 11 and 33 ft. on the Arkansas City gauge, the area of cross-section of the river in this vicinity in 1879 corresponded closely to that which is shown by Mr. Starling in 1889, and that in both cases these areas were less than are shown by the

Mr. Townsend. observations of 1885 and 1893. The year 1874 was a great flood year corresponding to 1884, and was also followed by a similar period of comparatively low water. This would indicate that at this locality during large flood years there is an enlargement of channel section, followed by a diminution of section during low-water years.

Annual surveys have been made of Lake Providence Reach during low water. The mean results of these surveys are graphically represented in the "Report of the Mississippi River Commission for 1892." The low-water areas appear to have followed a similar law to that of the area of cross-section at Arkansas City. A large increase in recent years in the width of river at bank-full stage is also to be noted. At the same stage, the discharge at Wilson's Point appears to have increased since 1890. Professor Whiting, by comparing recent surveys with the records of the Coast and Geodetic Survey, shows that there has been a perceptible enlargement of the river in recent years at New Orleans. Such evidence is not conclusive of a general law applying to the whole river. What may be termed accidental (unknown) conditions may have especially influenced these few localities. The entire river from the mouth of the Arkansas River to Vicksburg has recently been resurveyed, but the results have not yet been published. An examination has, therefore, been made of the relations which exist between the gauges which have been established at various localities.

An investigation made of all low-water readings from 1872 to 1892 of the gauges at Cairo, Helena, mouth of White River, Lake Providence and Vicksburg will be found tabulated in the "Report of the Mississippi River Commission for 1894," page 2971. That table shows a general tendency of the Helena gauge to relatively lower reading since 1882 than previously existed, when comparison is made with the Cairo gauge. The same is to be noticed at Vicksburg. At White River, on the other hand, there was a tendency to higher readings relatively to Cairo in recent years, while at Lake Providence there was first a rise in gauge readings, followed by a fall. During the low water of 1894 the gauges at Helena and White River bore the same relation to each other as existed since 1887. The great change in slope which accompanied this low water occurred between Cairo and Helena.

The gauges which have been established on the Mississippi River are very delicate indices of any change in its regimen, but the height which is recorded on a gauge is affected, not only by changes of area

Mr. Townsend.

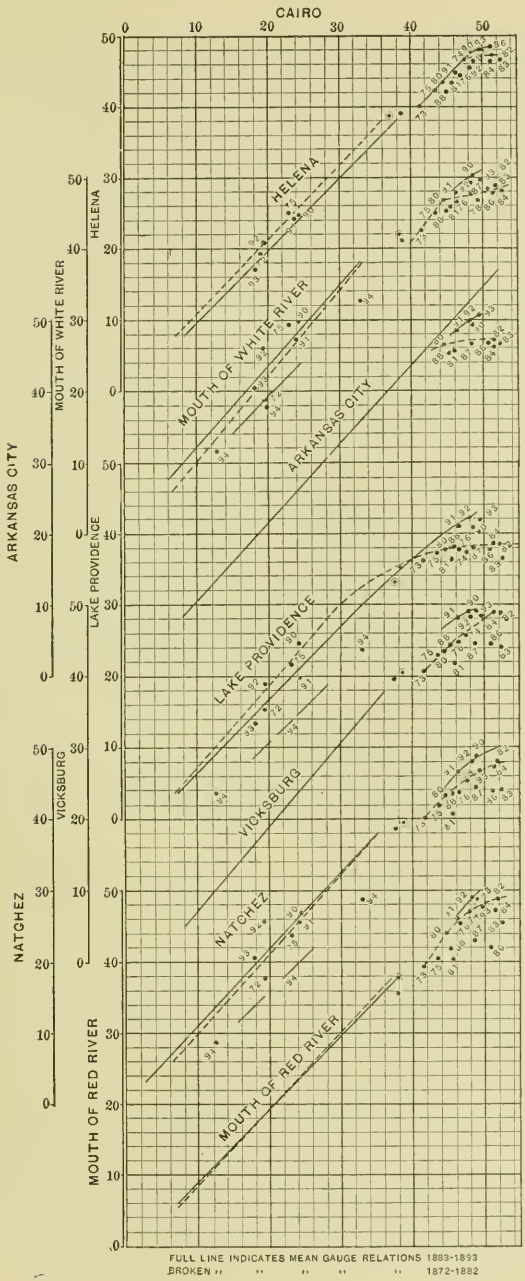

FIG. 4.

Mr. Townsend. of cross-section, but by any other cause which influences slope, such as the inflow of tributaries, the rate the river rises or falls, whether the observation is taken as the river is passing from a high to a low stage, or the reverse.

If there has been any change in the river section since levee construction has been resumed, the means of gauge heights for long periods before and after 1882 should show a difference; though large variations may be due to other changes in slopes, the means should show a residual which has been produced by the permanent causes acting during the periods under consideration.

Acting on this hypothesis, the heights on the gauges which have been attained by the crests of rises which have passed Cairo have been tabulated in groups varying by 5 ft., from 5 to 35 ft., and divided into two sections, from 1872 to 1882, and 1883 to 1893. The heights which these rises have attained at the principal gauges from Helena to the mouth of Red River, the portion of the river where levee construction has been most active, have been similarly treated. The means of the various groups have been plotted and connected by lines shown in Fig. 4. These lines do not represent the true gauge relations which exist at any time between Cairo and the gauges mentioned, but means about which the readings oscillate, and are termed mean gauge relations.

The results may be summed up as follows: That the crest of a rise which passes Cairo at 10 ft. on the gauge will tend to pass Helena 2 ft. lower since 1882 than formerly. At the mouth of the White River it will tend to be 1 ft. higher; at Lake Providence, 1 ft. lower; at Vicksburg, 3 ft. lower; at Natchez, 1 ft. higher, and at the mouth of the Red River will have about the same height. The crest of a rise of 30 ft. at Cairo now passes Helena at least 2 ft. lower than prior to 1882, the mouth of White River at the same height; Lake Providence, 3 ft. lower; Vicksburg at about the same height as it did in 1872, and about 2 ft. lower than it did from 1876 to 1880; at Natchez and the mouth of Red River it will attain about the same height.

The gauge relations submitted for Vicksburg prior to 1882 are, however, not satisfactory. A cut-off occurred in 1876 at Delta Point, opposite that city, which made a great change in the regimen of the river in the vicinity. The crests of rises which passed Vicksburg from 1872 to 1876 are too few for a good average, but appear to indicate

TABLE C.—Number of Days in the Year in which the River is at Various Heights.

Year.	Cairo Gauge.						Little Rock Gauge.						Jacksonport Gauge.					
	0-10 ft.	10-20 ft.	20-30 ft.	30-50 ft.	40-50 ft.	50-60 ft.	0-6 ft.	6-12 ft.	12-18 ft.	18-24 ft.	2 4-30 ft.	30-36 ft.	0-6 ft.	6-12 ft.	12-18 ft.	18-24 ft.	24-30 ft.	30-36 ft.
1872..	158	136	54	18	..	..	242	71	26	17	9	..	295	42	19	10	..	..
1873..	95	78	104	74	14	..	224	109	22	10	..	..	247	41	34	31	11	1
1874..	117	116	33	54	45	..	226	77	46	16	..	..	229	33	29	35	34	4
1875..	40	128	83	54	50	..	224	99	38	8	..	..	208	86	37	29	5	..
1876..	44	95	69	97	61	..	178	118	36	24	10	..	155	69	53	42	42	5
1877..	83	108	89	82	3	..	171	98	69	17	10	..	270	31	29	26	9	..
1878..	73	80	160	52	..	..	..	..	..	..	..	..	..	..	..	..	..	..
1879..	69	167	71	58	..	..	315	42	8	..	..	..	309	54	2	..	..	..
1880..	66	89	84	87	40	..	296	53	17	..	..	..	230	44	46	35	11	..
1881..	56	66	116	87	40	..	187	138	39	1	..	..	121	184	26	25	9	..
1882..	50	89	28	93	96	9	116	163	63	16	13	..	152	94	56	39	18	6
1883..	51	87	100	77	36	14	4	245	49	26	11	..	53	120	94	66	27	5
1884..	42	118	92	57	44	13	47	214	71	19	12	3	189	78	46	29	18	6
1885..	7	137	161	60	..	..	83	118	109	41	14	..	200	116	22	15	9	3
1886..	98	61	130	50	18	8	166	167	32	..	..	..	269	39	25	31	1	..
1887..	161	80	52	28	44	..	228	124	13	..	..	..	288	41	19	13	4	..
1888..	27	113	178	30	18	..	132	176	46	15	..	..	252	64	43	11	5	..
1889..	59	108	180	18	..	..	44	220	82	21	..	..	259	96	3	7	..	..
1890..	18	135	80	62	70	..	34	146	131	48	6	..	94	68	47	61	87	8
1891..	73	123	72	31	66	..	76	156	109	24	..	..	169	84	81	31	..	..
1892..	104	61	73	62	66	..	..	230	75	38	18	5	163	102	35	39	25	2
1893..	145	64	50	45	61	..	86	179	71	23	6	..	211	42	55	32	21	4
1894..	151	92	98	24	..	..	173	115	50	22	3	..	209	43	39	34	30	..

Average below 6 ft., Little Rock gauge, 1872 to 1882, 213.
Average below 6 ft., Little Rock gauge, 1883 to 1894, 75.
Average above 18 ft., Little Rock gauge, 1872 to 1882, 13.
Average above 18 ft., Little Rock gauge, 1883 to 1894, 27.

Average below 6 ft., Jacksonport gauge, 1872 to 1882, 22.2.
Average below 6 ft., Jacksonport gauge, 1883 to 1894, 19.7.
Average above 18 ft., Jacksonport gauge, 1872 to 1882, 42.0.
Average above 18 ft., Jacksonport gauge, 1883 to 1894, 52.0.

Mr. Townsend. some such gauge relation as above stated. When the cut-off occurred there was an abrupt change in gauge relations, such as would have occurred if the gauge had been moved up the river several miles, but retaining its zero line at the same elevation above the Gulf.

Before coming to the conclusion that these variations in gauge relations have resulted from changes in sectional area, it is advisable to investigate the other causes which can influence slope. The first to present itself to the mind is the in-flow of a tributary stream. Table C gives the number of days yearly that the Mississippi at Cairo, the Arkansas at Little Rock, and the White River at Jacksonport, have recorded the different heights stated in the headings. It will be seen from the table on the preceding page that in recent years there has been a large increase in the discharge of the Arkansas River, and a perceptible increase in the flow of the White River.

While this increase in the discharge of tributaries might account for the rise of plane at White River, it cannot account for the lowering of the plane at Lake Providence. A perturbation produced by the discharge of a tributary is regularly propagated from gauge to gauge, but could not cause one gauge to record a reading higher than the normal, and the next a lower reading, unless it exerted some influence on the river section.

The extreme low-water readings which are found at the different gauges in 1894, a year in which the river did not exceed a bank-full stage, would indicate that the lowering of gauge relations might be a low-water phenomenon with which levees had nothing to do. The years 1890 to 1894 afford an excellent opportunity to test this theory. At Lake Providence from 1890 to 1894 the gauge annually attained heights differing by about a foot, and though the table at Cairo would indicate little difference in the period the river was above the overflow stage, due to the influence of the Arkansas and White Rivers, the length of time the river at flood stage was exerting an influence at Lake Providence was greatest in 1890, then follow 1892, 1891, 1893, and last 1894, when there was no flood. The length of time the river was at a low stage at Lake Providence during these years is the reverse, 1890 being the shortest period, and 1894 the longest. On the curves of mean gauge relations at Lake Providence and at the mouth of the White River are plotted, in Fig. 4, the actual relations that existed during these years while the river was passing from a low to a high

Mr. Townsend.

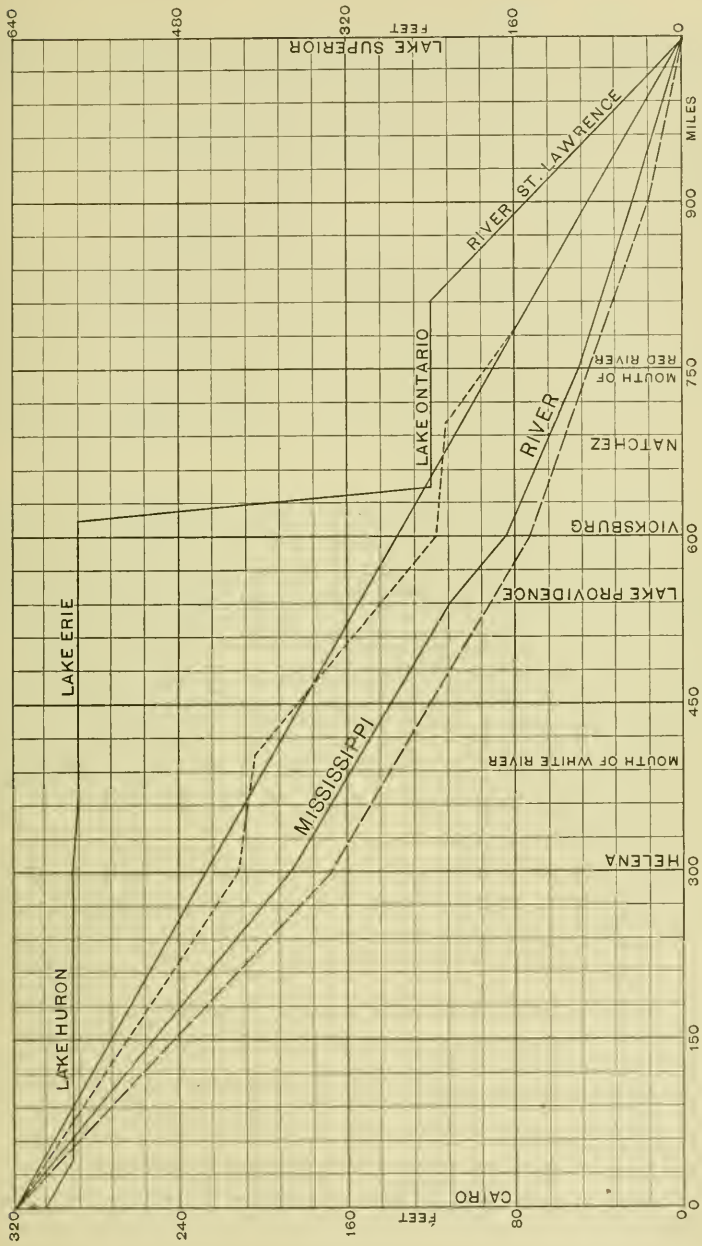

FIG. 5.

Mr. Townsend. stage, for crests of rises approximating 20 ft. An examination of these actual relations shows that the lowering of the readings has varied directly with the period of low water at Lake Providence. By a similar process of reasoning with the actual gauge relations that existed in 1872 and 1875, the years in which the river remained the longest and least times respectively below 10 ft. at Cairo prior to 1882, a similar result is obtained.

The conclusion is obvious that low-water conditions can eliminate or obscure the results due to high water, but if actual gauge relations prior to and after 1882 are compared, it will be seen that the same forces acting in a river at a low stage can now cause a greater lowering of the gauge at Lake Providence than formerly, while at the mouth of the White River the influence is not as great. This is clearly exemplified by comparing the actual gauge relations of 1893 and 1872, and of 1875 with those of 1890 and 1891.

If the actual relations existing as a river is passing from a high to a low stage are compared, no relation is apparent between the height that a gauge reads and the period the river is above an overflow stage, but it will be found to bear a definite relation to the length of time the river is passing from high to low water. The influence of high stages is obscured by medium stages, but, as with low stages, the medium stages produce in recent years greater results at Lake Providence than at the mouth of the White River. The lines of the diagrams, therefore, give residual results, and are true indices of changes which have taken place in the regimen of the river since levee construction has been resumed.

If a 20-in. main conveyed water from a reservoir at Cairo to the Gulf of Mexico, a hydraulic grade line would be established which is represented by the right line in Fig. 5. If, however, the sections between Helena and the mouth of White River, and Vicksburg and Natchez, were removed, and 30-in. pipe substituted, the hydraulic grade line would not remain the same, but would take some such position as is indicated by the broken line. In a river, the same results would be attained, if between the same localities there had been a diminution in resistance to flow such as would occur from an increase in area of cross-section; at the upper end of the improved reach the water would move away faster than it formerly did, while it could not escape from the lower end any faster until it had acquired head suffi-

cient to overcome the resistance in the unimproved portion below. Mr. Townsend.
The rise in plane noted at the mouth of the White River and at Natchez is just as indicative of relative reduction in resistance to flow and possible enlargement of section as the fall in plane at the other stations, but it shows whatever enlargement has taken place in the river has only been partial; that some reaches have been more affected than others. If the remainder of the 20-in. pipe were replaced by 30-in., the hydraulic grade line will then change from the broken line back to the straight line. The 30-in. main will be under exactly the same pressure at every point as was the 20-in., but it will discharge into the Gulf much more water; similarly, if a river were connected with a large lake at Cairo, and its channel were uniformly enlarged, the result would be to discharge more water, but the hydraulic grade line would be unaffected.

It may be objected to this process of reasoning that there is no lake at Cairo, and that the straight line is not the hydraulic grade line of the Mississippi River. The writer has plotted on the same figure the flood heights at the different gauge stations in 1893, and connected them by a line which will approximately represent the existing hydraulic grade line of the river during floods. The reason this line does not conform to the right line in the diagram is because there is not a uniform resistance to flow from Cairo to the Gulf. As explained in the paper, in the lower portions of the river the channel offers less resistance to discharge than above. Returning to the illustration of a pipe line, the large pipes have been concentrated at the lower end of the river. In a river system where the sections offering the least resistance to flow are at the upper portions of the river, as exists in the St. Lawrence, the hydraulic grade line assumes a position concave to the axis of distances instead of convex, as is the case with the Mississippi.

It is extremely desirable to reduce flood heights from Helena to the Gulf, that is, to establish a grade line represented by the lower broken line in the diagram, but it is self-evident that to attain such a result, the slope from Cairo to Helena must be increased to compensate for the reduction in slopes below Helena, if the river attains the same height in the two cases at Cairo. To obtain such a result, the river must remain as it is from Cairo to Helena, and be allowed to enlarge its channel, if possible, from Helena to the Gulf. If an opposite

Mr. Townsend. course is pursued, and the river is enlarged from Cairo to Helena faster than it enlarges below, then the slope between Cairo and Helena will diminish, more water will be delivered at Helena than the unimproved river below can carry off, except under increased head, and the relative gauge readings in the lower river will be increased.

The approximate hydraulic grade line of the St. Lawrence River from the mouth of Lake Superior to Montreal is added to the diagram to illustrate what would result if resistance to flow from Cairo to Vicksburg was reduced to a minimum and concentrated between Vicksburg and the Gulf, if the effective head which exists at Cairo could be delivered at Vicksburg.

The only other assumption requiring examination is that the river might retain the same height at Cairo if its channel should be enlarged. During low water it is evident that it could not. Under present conditions all the water is discharged at Cairo that is received; if not, the river would rise. If, however, the lower river was enlarged and the resistance to flow diminished, it would discharge a greater quantity of water under the same head than at present. To supply this discharge would require greater velocities in the upper rivers than now exist, which could only be obtained by increasing the slope, viz., by lowering the Cairo gauge. During high water a very different state of affairs exists. This enlargement of section from Helena to Cairo is to result from the construction of a levee line, which will prevent the inflow of water into the St. Francis Basin, and in order to reduce flood heights at Cairo, there must result an enlargement of channel section which will compensate for the unrestricted flow of the crests of flood waves into a basin 40 miles wide, containing an area exceeding 6 000 square miles.

The author proves that higher rather than lower flood heights are to be apprehended at Cairo with a completed levee system. The gauges indicate the same thing. The levee construction in the basins below the mouths of the Red and White Rivers and below Helena have not been accompanied by lower readings of the gauges at those localities during flood stages.

The essential element it is sought to emphasize by this discussion is that the height water will attain is primarily a function of the effective head to which it is subjected; that in a pipe, a sewer, a big ditch or a little ditch, if the pressure on opposite sides of a water film are equal in the different cases, it will rise to the same height independent

of the area of the section or the character of the surrounding medium. Velocity of discharge can not affect flood heights, except by influencing the effective head, and this is a function of the relative velocities above and below the discharge section, rather than the mean velocity of the entire river. Mr. Townsend.

Electrical engineers have a tendency to compare electricity to water. Hydraulic engineers can much more profitably reverse the process, as the electrical engineer has solved many of his mathematical problems, while the hydraulics of rivers shows only an array of failures. The electrical formula, $C = E \div R$, gives promise of better results when applied to rivers than any modification of $v = c \sqrt{r s}$, though trouble will probably be experienced in determining R. With an electrical current, by increasing the size of the wire, thus reducing the resistance R, the quantity of electricity discharged is greater; if the size of the wire is so increased as to short-circuit the battery, the electromotive force rapidly runs down. In a river, an increase in cross-section increases discharge as long as the head can remain constant, but a large increase in cross-section short-circuits our battery and reduces the head. That is what takes place in an unleveed basin during floods.

The heights attained by floods at the different stations have been plotted in Fig. 4, and the curved lines inclosing them show the maximum limits that floods have attained at the different stations under varying heads at Cairo. At all the stations prior to 1882 there was a tendency of these limiting curves to become tangent to some horizontal line; that is, with the river unrestrained there is a certain height that cannot be exceeded at the station, no matter what height the river attains at Cairo under the conditions which have heretofore existed. This cutting off of extreme flood heights that is so noticeable in the different diagrams is attributable to the reservoir capacity of the different basins. Their influence may be compared to that of a standpipe in a water system, or of the air-chamber of a pump, which tends to equalize the variations of pressure. The greatest pump pressures can not be transmitted beyond the air chamber.

The curves limiting the floods since 1882, in all cases except Vicksburg, show a rise, and the tangency to horizontal lines is also less apparent, that is, levee construction, by reducing the reservoir capacity of the river, has already markedly raised flood heights. As the reservoir capacity of the basins is still further reduced, these limiting curves

Mr. Townsend. will still more closely approach the line of gauge relations below the overflow stage, and if the reservoir capacity be reduced to that of the river channel, these curves will either correspond to the lines of gauge relations below the overflow stage, or pass beyond them, for it must be borne in mind that the lines below the overflow stage express mean conditions, while those above that stage express maximum.

It is difficult to predict what the people of a district are going to do, but as most of the land under cultivation in the St. Francis Basin is close to the river bank, the temptation of the levee boards will be great to repeat the methods of the lower districts, and place their levees as close to the river banks as possible, restricting the reservoir capacity of the river practically to that of its channel. Under such conditions the line of gauge relations below the overflow stage may be expected to be extended to the extreme height attained by the flood at Cairo.

The changes which have taken place in the relations existing between gauges at stages varying from 35 to 40 ft. confirm both the deductions which have been presented as to changes in gauge relations below the overflow stage, and also during floods. In discussing the gauge relations which exist below the overflow stage these gauge heights were omitted, as they are affected not only by changes in the channel but by the escape of water through sloughs or bayous below the overflow stage, a rather indefinite expression when applied to the Mississippi River. They have been added to the diagram of gauge relations (Fig. 4), however, the mean value prior to 1882 being represented by a dot, since 1882 by a dot inclosed in a circle. Wherever the gauge relations, both above and below the overflow stage, show a rise in plane since 1882, as at the mouth of the White River, a marked rise in the mean gauge relations between 35 and 40 ft. obtains. If, however, there is a lowering of the plane below bank-full stage, accompanied by a rise in plane above, as at Helena and Vicksburg, very little change occurs in these intermediate gauges. At Lake Providence a lower mean-gauge relation is found since 1882, but markedly less than occurs between 30 and 35 ft. on the gauge. At the mouth of the Red River where little change in gauge relations occurs below the overflow stage, the rise in plane of these gauge relations is especially marked. At Natchez the same law is observable, but less pronounced than in any of the other cases.

The author has discussed the influence of the reservoir capacity of

the river channel on discharge. Combined with slope it also produces marked influence on gauge heights, variations of 3 ft. from the mean gauge relations being not infrequent, and the gauges indicate the conditions favorable to a lowering of gauge readings from these causes. These are, a rapid rise at a period when the lower river is at a low stage, followed by a rapid fall; but if the conditions are reversed, if the lower tributaries happen to be high when a gradual rise appears at Cairo, the influence of the reservoir capacity of the river is slight. Mr. Townsend.

It has been argued that because the maximum discharge in the river channel at Lake Providence in 1893 exceeded that at the same locality in 1882 by 50%, with an increase of flood height of 3½ ft., that 100%, or the entire flood discharge of the Mississippi River in 1882, could be carried by Lake Providence with a further increase of 3½ ft. Under favorable conditions such might possibly be the case, but the levee engineer is not seeking favorable conditions; it is the unfavorable that worry him. After a night spent in watching his levee line while a wind is driving waves over the tops of his levees, he views with considerable relief a bright, calm morning. He desires to know, not the least height at which a given discharge will pass, but the maximum height which is to be apprehended under the most unfavorable conditions that are liable to occur.

From the preceding discussion of gauge relations the writer arrives at practically the same conclusions that the author obtains by discussing discharge observations; that existing levees are far too weak to resist a great flood in a river restrained from Cairo to the Gulf; that with the reservoir capacity of the St. Francis Basin undiminished floods are liable to occur which will strain existing levees to their utmost capacity, and that the influence of enlargement of section on flood heights will be local. The danger, however, to the inhabitants of the lower basins is not as imminent as this discussion would appear to indicate. It is a safe prediction from experience with other basins that the next great flood will not be entirely prevented from entering the St. Francis Basin. There is also a safety-valve at the head of the Tensas Basin which will prevent abnormal flood heights below. An increase in flood heights at Arkansas City will be accompanied by a discharge around the head of the Tensas levee system. Before the gauge could record 56 ft. the area of the waste weir would equal that of the discharge section, though obstructed by forest growth.

Mr. Starling. WILLIAM STARLING, M. Am. Soc. C. E.—Since the publication of the paper, the author has received several interesting communications bearing on the history of river gauging in the United States. The earliest example of a telegraphic meter, and also the first instance of a meter of the anemometer type, was the device of D. Farrand Henry, M. Am. Soc. C. E., then United States Assistant Engineer in the service of the Lake Survey. In 1868, Mr. Henry constructed a current meter out of an old Robinson's anemometer, and adapted to it a recording apparatus consisting of a break-circuit arrangement and a Morse register. He also used a meter of the screw or propeller type, with an electrical connection, and thought it the preferable form. With these instruments a large number of very interesting observations were made on the St. Clair and Niagara Rivers. Among others, an extensive series of measurements was carried out to determine the distribution of velocities in vertical planes, with the primary intention of finding out a coefficient for double floats, in the use of that method. The results of these measurements agreed well among themselves, but were quite different from those obtained by floats, and the curve of vertical velocities did not correspond to the parabola of Humphreys and Abbot. In the "Reports of the Chief of Engineers" for 1868 and 1869, and again in the report for 1870, Mr. Henry gave an account of these experiments and his reasons for preferring the meter to double floats. These reasons are given in detail in the pamphlet published by him in 1873,* which contains a valuable summary of the engineering knowledge at that date on the subject of the distribution of velocities in vertical planes, and also on that of the rating of meters.† These observations were referred by the chief of engineers, General Humphreys, to General Abbot, and were reviewed by the latter with much asperity.‡ Mr. Henry replied in a similar tone in a supplement to his pamphlet, wherein he maintained his position with ability, and gave some interesting observations on the theory of double floats, by which it would appear that at considerable depths they must always give a velocity in excess of the true rate.

The rating of these early instruments was performed very much as at the present day. The rating line found, as had been observed by

* "The Flow of Waters in Rivers and Canals," by D. Farrand Henry, Chief Engineer Detroit Water-Works, Detroit, 1873.

† See also the "Report of the Chief of Engineers" for 1870, p. 554.

‡ "Report of the Chief of Engineers" for 1870, p. 616.

Baumgarten and Boileau, was a curve, which, at high velocities, became almost a straight line. Mr. Starling.

The Ellis meter afterward used (in 1874) was constructed on the same principle as the anemometer meter of Mr. Henry, to whom, indeed, General Ellis acknowledges his obligations. It was used in conjunction with a meter of the Woltmann type. Both were rated by the equation of a straight line. Both the observations of Mr. Henry and of General Ellis* are well worthy of the attention of those who are interested in this part of hydraulics.

During the month of October, in 1879, a notable series of observations was made on the upper Mississippi, near Burlington, Ia., under the direction of Alexander Mackenzie, M. Am. Soc. C. E., by G. A. Marr, M. Am. Soc. C. E. This work appears to have been executed with much care and thoroughness. Six meters were used at once, placed at equal distances apart on a wire cable, suspended from a derrick which projected some 20 ft. in front of the bow of an anchored boat. The depth of water varied from 11 to 27.6 ft. The details of the processes pursued will be found in Colonel Mackenzie's report, published several years afterward.†

An interesting fact, established both by Mr. Henry's and Mr. Marr's observations, is the existence of very decided pulsations of the current, alluded to in the paper.‡ Mr. Henry found lesser fluctuations of half a minute to one minute in duration, with maxima every five or ten minutes. They did not seem to be synchronous with the oscillations of the surface level, which were also very irregular. "Often," says Mr. Henry, "when two floats were put out from the upper boat, one minute apart, the last one would gain upon and sometimes pass the first." It was found that the pulsations were much the greatest near the bottom, where the minimum velocity observed was sometimes less than half the maximum.

The observations are corroborated by Mr. Marr, who also remarks that the changes generally occur at about the same time from the surface to the bottom. In confirmation of these statements, very interesting plottings are given by Mr. Marr. The pulsations are not capricious, but occur with considerable regularity, and with approxi-

* "Report of the Chief of Engineers" for 1878–79, Vol. I, p. 312.

† Report on Current-Meter Observations in the Mississippi River, near Burlington, Iowa, by Major Alexander Mackenzie. Washington, 1884.

‡ *Transactions*, Vol. xxxiv, p. 385.

Mr. Starling. mate periodicity. The periods of the major and minor fluctuations correspond closely with those found by Mr. Henry. So also does the amount of the oscillation, which, according to Mr. Marr, is sometimes nearly half the greatest velocity, in less than one minute of time.

The value found by Mr. Marr for the relation between mean and mid-depth velocity is exactly the same as that reported in 1882 at the stations on the lower Mississippi, viz., 0.958. General Ellis found it 0.940. The ratio of the depth of the fillet of mean velocity in vertical planes to total depth is 0.622, against 0.63 in the lower river. Mr. Henry had found it to be about 0.6. General Ellis had given it as 0.636. Mr. Price has remarked that the observations of 1882 cannot be implicitly relied on at some of the stations. This is doubtless correct, yet the mean results may probably be accepted as fair approximations.

Professor Spalding takes a somewhat sanguine view of the situation on the lower Mississippi, with which service he was once connected, and thinks that the estimates given in the paper of flood heights eventually to be attained may be excessive. This is the earnest hope of all who are engaged in the levee service, and there is some ground for it. Observation so far as it goes, is indecisive, having been continued for so short a time. A few words will be said on this point hereafter.

Professor Spalding thinks that a straight line may be made to fit the ratings of meters within the ordinary errors of observation. Perhaps it could, yet it would be but a misfit after all. Because there are errors which are unavoidable in observation, it does not follow that additional mistakes should be voluntarily incurred by faulty methods of interpretation. It is shown by Mr. Price in his discussion of the paper that the equation of the rating curve varies with the form of the wheel. Mr. Henry found the curve to be an ellipse whose axes were 8.28 and 3.44, respectively. Mr. Marr found it to be a straight line.

Mr. Crowell's remarks about the diminution of velocity in the stream by the transportation of sand and gravel are doubtless correct, though it is not easy to estimate this influence quantitatively. It is thought, however, that this should not be confounded with viscosity. There is no such thing as "thick mud" in the flowing water of the Mississippi, the quantity of matter in suspension being probably never sufficient to impair the fluidity of the water to an appreciable extent. The highest recorded ratio, by weight, of sediment to water is 1 to 195, or something more than half an ounce to the gallon. This is a solitary

case. The next highest is 1 to 549, or about a fifth of an ounce to the gallon. In the Missouri, the remarkable proportion of 1 to 37 has been observed, or nearly 4 ounces to the gallon. Mr. Starling.

Mr. Dabney calls attention to the omission by the author of the effect of bank erosion on the load of sediment carried by the stream, and its influence on bar-building. The point is very well taken. There seems to be no question that the material which goes to form troublesome shoals is very largely derived from the caving reaches in the bends or pools above, and this effect should be added, in many situations, to that of the regular scour which takes place during every flood in the deep places. In other instances, the erosion of the bank seems to be merely the consequence of what is taking place on the bottom. It is the latter which scours, and the bank falls in, being undermined, to replace the material which has been removed from near its base. On the other hand, at low water, it is the shoal which cuts out, and this cutting out is often confined to a narrow section near the bank*, inducing a low-water cave. In the case of the discharge sections quoted in the paper, there were no caving banks of any consequence in the vicinity, the sections being chosen partly on account of their freedom from this disturbing influence.

Mr. Harrod's remarks on the effect of the confinement of the flood waters by levees upon the low-water line are in strict consonance with the evidence. The progressive decrease which has been observed since 1890 of the gauge readings below Cairo still continues, the low water of 1895 having been about 1 ft. below that of 1894, which was itself the lowest on record up to that time, from Memphis to Vicksburg. By those dependent on levees, the steady decrease of low-water heights is very naturally attributed to the confinement of the flood discharge with which it was coincident in time; and this the more that such a result had been anticipated by them as a consequence of the completion of the levee system. In this view the author is inclined to concur; yet, as Mr. Harrod says, the period of observation is too short to allow of a positive conclusion, and there are other agencies which might have had a potent effect in producing the phenomenon in question, One of these is the very prolonged period during which low water has prevailed for the last three or four years. The average length of the period of low water, during which the river has been below the stage

* *Transactions*, Vol. xxxiv, p. 472.

Mr. Starling. of 12 ft. on the Helena gauge, is 97 days. The greatest periods have been, in 1872, 165 days; in 1874, 135; in 1886, 131; in 1887, 170; in 1892, 110; in 1893, 175; in 1894, 177; in 1895, 265 days. The natural tendency of the low-water stage is to cut out the bars, and this process would reduce the low-water line as effectually as a general enlargement of the cross-section.

Though the low-water mark be reduced in height between tributaries, whether by cutting out the bars or by enlargement of the section, yet it does not necessarily follow that the high-water line must be lowered. If there be an increase of section in the direction of width, and at the same time a decrease of mean depth, which is perfectly compatible with an increase of maximum datum depth, the flood-line may not be reduced in height at all. Mr. Harrod has shown* that the net result of continued caving is an increased width of river; and Captain Townsend's surveys of the Lake Providence Reach in 1891† seem to point in the same direction.

While all levee engineers, therefore, hope for a reduction of flood-heights as a consequence of the confinement of the waters of the river to its channel, and while they think they have good grounds for such a hope, they do not like to be too sanguine of such a result, and they are certainly not justified in relaxing their efforts on the faith of such a prospect.

Mr. Price thinks that the cleaning of a meter or the oiling of a bearing has but little effect on the rating. The opinions expressed in the paper were derived from the experience of observers. At Point Pleasant, in 1885, on February 3d, the meter, when turned by hand, took 78 seconds to come to rest. On the 5th of February it required 170. The explanation given is that the cup containing the electrical connection had been filled with oil in the meantime. On March 27th a new contact apparatus was put on, changing the time required to come to rest from 185 to 265 seconds. In 1891, at Wilson's Point, it was found that the rates of the same meter were very different. In seeking to explain these differences, Mr. T. C. J. Baily writes:‡

"The larger values of the coefficients a and b for Meter No. 5 found at Huntington from those found at Wilson's Point I think are

* Paper on the protection of banks, in the "Report of Mississippi River Commission" for 1885, p. 2885.

† "Report of the Mississippi River Commission" for 1893, pp. 3786–3806.

‡ "Report of the Mississippi River Commission" for 1892, p. 3124.

due to the fact that at the former place only the lower bearings were oiled, for fear of injuring the electrical connection, while at the latter place both bearings were oiled. * * * I have found that the stiffness of the oil used has a marked effect upon these coefficients. * * * A very marked difference appears in the rate of Meter No. 5 after it was reported as being thoroughly cleaned." Mr. Starling.

Indeed, it might be inferred from Mr. Price's own remarks that friction plays an important part in determining, not indeed the form of the curve, but its position with reference to the origin of co-ordinates, or the constant *b*. However, this is a subject on which Mr. Price ought to be classical authority.

Mr. Miller's remarks on the advisability of selecting wide rather than narrow places as discharge sections are worthy of attention, especially as proceeding from an observer of long experience. What he says about the change of location of the sections at Arkansas City and at Wilson's Point in 1893 may partially explain some of the discrepancies which had been observed at those points in previous years.

Mr. Coppée's instance of a remarkable pulsation of the flow of the river corresponds to the experience of all engineers who have had occasion to observe the behavior of the Mississippi or other large streams in this respect. Any one who undertakes to get the reading of a gauge, even on a still day, will find that the water level is in a state of continual fluctuation, and it is well nigh impossible to define it with precision. These fluctuations do not necessarily correspond in time, as Mr. Henry has remarked, with the pulsations of velocity.

Professor Johnson thinks the study ought to have been carried a little farther, and desires estimates of the heights to be attained in other portions of the river. In most investigations made by practising engineers, it is not the mere abstract love of scientific truth that is the impelling motive, but some immediate need. In the present instance, the object sought was to fix the ultimate grade of levees in the author's district, of which Arkansas City and Wilson's Point are principal stations. The investigation was not extended further, principally from the lack of personal familiarity with the local conditions existing elsewhere, and from the great additional expenditure of labor and time which it would have involved. It was thought that if the methods pursued in the paper stood the ordeal of criticism and appeared to be

Mr. Starling. sound, the scope of the investigation could afterward be extended so as to embrace other parts of the river.

At Red River the situation seems to be very complicated. That stream has acted, at different times, both as an inlet and as an outlet. It was formerly the principal channel through which overflow waters from the Tensas Basin were returned to the Mississippi. Testimony as to the facts is not harmonious. The majority of the Mississippi River Commission think that with the exclusion of the return flow from the Tensas Basin consequent on the complete closure of that front by levees, Red River would act as an outlet, and that the shutting-off of it would cause a considerable increase in the discharge to be carried by the Mississippi below its confluence.

On the other side, Colonel Suter, who has been studying the Atchafalaya problem for twelve years or so, maintains that the flow from Red River into the Mississippi is "the great factor in disastrous floods in lower Louisiana, whether the water producing this inflow be crevasse water from the Mississippi or the drainage from the Red River Basin." "The Atchafalaya," he continues, "never has acted as an outlet in great floods, and I do not believe that it would do so, even if the crevasse water were excluded, as the Red River is invariably in flood at such times."

One would not care to give an opinion on such a vexed and important subject without careful study and a personal familiarity with the facts and the localities, which is not possessed by the author.

Mr. Hider's detailed account of the methods taken to insure accuracy in the practical work of measuring the discharge is very instructive, and shows conclusively that it was not the fault either of the instruments or of the men that discrepancies occurred between the discharges reported at two adjacent stations. The candor with which these discrepancies were reported and published, without any attempt to cook them, is an excellent evidence of the true professional spirit which actuated the engineers engaged. The method of integration in a horizontal plane, proposed and executed by Mr. Hider, is one that seems to have the elements of great promise in it. The greatest drawback to it is the necessity of an assumption of a regular distribution of velocities in vertical planes. Some such assumption is necessary, however, in most of the methods which have been yet employed.

Captain Townsend's discussion brings into the field the interesting but not at all simple question of the enlargement of section of a more or less permanent nature and of a general kind, and the resulting influence on gauge heights at low water and in flood. The paper has barely touched this subject, partly because the evidence, so far as it is published, is scanty; and partly because the testimony of the gauges does not show any material alteration in the position of the flood line at the points where the enlargement is known to have occurred. The evidence adduced by Captain Townsend of the changes brought about in the gauge relations at low and medium stages appears incontrovertible. These changes, however, seem to be confined mostly to such stages. At Helena, for instance, Captain Townsend shows * that at low water the gauge at Helena, as compared with Cairo, is 2 ft. lower since 1882 than before that time. So, also, the crest of a rise reading 30 ft. on the Cairo gauge passes Helena at least 2 ft. lower after 1882 than before. Captain Townsend's tabulations extend no higher than about 35 ft. The author has carried them up to 40, at which stage overflow barely begins. At stages between 35 and 40 ft., it is found that the Helena gauge, prior to 1882, stood at an average of 0.84 ft. above the Cairo gauge, since 1882 at an average of 1.06 ft. above, showing that under these circumstances the crest of the wave now passes Helena at a slightly higher stage than formerly. Mr. Starling.

It does not seem very profitable to compare Lake Providence with Cairo, for the two great southern tributaries, the White and Arkansas Rivers, intervene, and pour out floods which are sometimes of enormous volume, and are so variable and capricious, and influence gauge heights below them to such an extent (sometimes several feet) as to obscure altogether the ordinary relations. It would be very proper, however, to compare Lake Providence with Arkansas City, which is below the mouths of the tributaries. Gauges have been kept at Arkansas City only since 1880. Since that time the relations between this point and Lake Providence have been as shown in the table on page 366.

The figures represent the differences between the gauge readings at the crests of rises at Arkansas City and Lake Providence, Arkansas City being the higher.

* "Report of the Mississippi River Commission" for 1894, p. 2972, Table No. 2.

Mr. Starling. RELATION BETWEEN GAUGE HEIGHTS AT ARKANSAS CITY AND LAKE PROVIDENCE.

Year.	0-10	10-20	20-30	30-35	35-40	40-45	45-50
1880....	+0.30	+3.0	+3.45	+4 30	+4.92	+3.95	+7.25
1881....	.....	.....	6.20	5.15	6.15	7.23	.....
1882....	1.40	2.01	3.65	.....	.....	7.74	8.85
1883....	.....	2.87	4.47	4.98	.....	7.95	9.83
1884....	3.87	3.87	5.08	5.20	6.00	7.65	8.35
1885....	.....	3.18	5.86	6.62	6.68	7.00	
1886....	1.21	2.14	5.74	6.00	6.62	7.20	8.99
1887....	0.24	4.26	5.46	.....	7.16	.....	8.65
1888....	.....	.....	5.96	6.37	6.90	.....	7.27
1889....	.....	3.67	5.45	6.61	6 78	.....	
1890....	.....	3.57	5.6	.....	5.9	.....	7.4
1891....	0.2	5.9	5.2	6.5	.....	.....	6.7
1892 ...	2.8	5.9	6.2	.....	.....	.....	7.0
1893....	3.3	4.8	5.7	.....	6.4	.....	7.2

The principal change which has occurred in the regimen of this part of the river since the beginning of the period embraced in the table is the rebuilding of the Arkansas levees in 1887. The effect of this work is seen plainly in the last column of the table, which shows the differences between the gauges at very high stages. This difference has been considerably reduced since 1887, from the confinement in the channel of large volumes of water which formerly escaped over the bank between Arkansas City and Lake Providence. It should be observed that the figures given in the last column, for the years since 1887, do not represent the actual difference between the heights of the crests of the rises at such times as there were breaks in the levees between Arkansas City and Lake Providence. Such a comparison would be of little profit or interest. The differences given are those existing at the time the first crevasse occurred, in each year, with a suitable time correction to allow for the passage of the crest from the one station to the other. For instance, in 1892, the first crevasse took place on May 9th, between Arkansas City and Lake Providence. On that day, the reading at Arkansas City was 48.8, and the river was nearly stationary. At Lake Providence the gauge was 41.6, and rising at the rate of about 0.1 ft. a day. It takes about a day and a half for the crest to pass from the upper to the lower station. Therefore the reading at Lake Providence corresponding to 48.8 at Arkansas City would have been about 41.75. So in 1893, the first break occurred on May 11th, between Arkansas City and Lake Providence, the gauge at the former station reading 49.0 and at the latter 41.4.

The river was rising at the rate of about 0.3 ft. per day. It is therefore inferred that about 0.45 should be added to the reading at Lake Providence to get the difference which would have prevailed for a crest of the given height. This would give 41.85, or a difference below Arkansas City of 7.15. By another process it was estimated that the stage at Lake Providence, at the height of the rise in 1893, would have reached about 42.8, if there had been no crevasses below Arkansas City. This would give a difference of about 7.4. Mr. Starling.

It is thought that these facts substantiate the position taken in the paper that "the relation between the gauges at Arkansas City and Lake Providence seems to have undergone no considerable alteration since 1885."

The table of maximum discharges at Lake Providence given by Captain Townsend for the different flood years in which they were measured, with the statement that an increase of discharge of 440 000 cu. ft. per second has been accompanied by a rise in gauge height of only 3.5 ft. requires some comment. It is well understood that Captain Townsend is not responsible for these figures nor for the inference that 3.5 ft. more would carry the whole discharge of 1882. It would be very pleasant to entertain this belief if there appeared to be solid grounds for it. It is feared, however, that it will not stand examination. The discharge of 1 057 000 cu. ft. at Lake Providence in 1882 was carried, not at the maximum stage of 38.32, but at 34.85, nearly three months later. There was carried indeed a discharge reported as 936 954 cu. ft. at the highest stage, but this was just at the time of the great return flow at Vicksburg,* whereby the slope Lake Providence-Vicksburg was reduced from the ordinary high-water slope of 0.30 ft. to the mile to 0.23 ft., thereby raising the gauge at Lake Providence by the effect of back-water; and moreover the discharge recorded does not include the flow over the banks outside of the discharge section, which was considerable, as the station was a mile or more outside the levee. In June, when the maximum discharge of 1 057 990 cu. ft. is recorded, the slope was exactly the normal for that stage, namely, 0.301 ft. to the mile, and there was no discharge worth mentioning over the banks. Hence the stage, 34.85, might properly be taken as corresponding to that discharge. The velocities were taken with meters. So in estimating the greatest discharge ever passed

* See *Transactions*, Vol. xxxiv, pp. 478 and 479.

Mr. Starling. within banks, that of 1893, it would not be proper to take the height actually attained, 41.8, reduced as it was by four considerable crevasses, all within the distance of 45 miles above Lake Providence, and apply to it the discharge obtained by adding to the observed discharge the flow through those crevasses. It would be more reasonable to take the stage and the discharge observed just before the first crevasse. These were 41.4 and 1 400 398. There would thus be an increase of discharge of 342 408 cu. ft. for 6.55 ft. of rise of the river. This result would be far nearer to the truth, yet even this would not be accurate, for the gauge relations between Lake Providence and Greenville, 64 miles above, in 1882, show that there was a disturbance at the former place, due to loss of water over banks and through crevasses either between Greenville and Lake Providence or below Lake Providence or both. It is useless to attempt to estimate these perturbations because there are no data sufficient for the purpose. Indeed it is time lost to reason from such premises at all.

Captain Townsend's inference, from accessible data, that general enlargement of the section at Arkansas City occurs during great flood years, and that contraction of the section attends years of low water, and especially a succession of such seasons, appears consonant to probability. It is not impossible that these changes are general throughout the river. There are some indications that such may be the case. If this be borne out by accurate and long-continued observation, it will lead to very interesting conclusions. If there is a general fill throughout the river during a succession of low-water years, it must proceed from the contributions of solid matter brought down by the tributaries, and it would appear plain that the main river, at such times, had not sufficient transporting power to carry to the sea these accessions of sediment; while in years of flood it had a superfluity of power, enough to remove the accumulations of previous years and perhaps to make new inroads on the banks and bottom. This would conform to the general idea that sedimentary streams enlarge their beds if they have to carry increased volumes; and that, on the other hand, as Captain Leach expresses it, that they will not tolerate beds which are too large for them. It would be very instructive to examine into the connection, if any, which these phenomena have with the stages of the Ohio. Great floods have usually proceeded, for the most part, from this stream, which is, comparatively, a clear-water river. Again,

even during low-water years, there have usually been large accessions, both of water and of sediment, from the Missouri and other muddy streams. Mr. Starling.

The diminished section at Arkansas City in 1879 and in 1889, to which Captain Townsend refers, might be accounted for by the ordinary fill in pools in low water; but this would not explain the high stage at which the discharge passed in 1889, or change of plane, which would indicate a fill on the bar as well as in the pool.

The enlargement of section at Wilson's Point seems to be well established. The following are the extreme measurements of the datum area from the earliest observations:

1884	172 129 to 195 504
1885	180 333 to 198 438
1887	191 139
1888	174 672 to 188 075
1889	164 648
1890	157 707 to 194 422
1891	191 391 to 222 272
1892	203 029 to 233 659
1893	197 988 to 229 292

It would thus appear that the datum area at this point has increased since 1884 some 25 000 to 35 000 sq. ft., and that most or all of this has occurred since 1890; and by far the greater part during the interval between the floods of 1890 and 1891. Notwithstanding this, the high-water gauge relations between Lake Providence, 11.1 miles below Wilson's Point and Arkansas City, where the section is nearly the same as in 1884, are almost or quite unaltered. In other words, the high-water discharge passes at about the same height as it would have done had there been no enlargement of section; and the increase of area is accompanied by a diminution of velocity. This is incompatible with a general enlargement of section below Arkansas City, unless it be entirely in the direction of width, and be attended by a decrease of mean depth, which indeed seems probable.

As Captain Townsend says, the levee engineer cannot afford to depend upon favorable conditions; it is the unfavorable that trouble him, and it is against these that he is bound to provide.

Mr. Starling. The views expressed relative to the changes to be produced by the leveeing of the St. Francis Basin seem to be sound. As this will soon be an accomplished fact, there will be an opportunity to bring the reasoning to the test of fact.

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No. 778.

THE STRENGTH OF PILLARS.—AN ANALYSIS.

By LEOPOLD EIDLITZ, Esq.

PRESENTED MARCH 4TH, 1896.

WITH DISCUSSION.

When a pillar is compressed endwise by a force gradually increased to the breaking point, it is reasonable to assume that at the intrados of the pillar at the point of greatest deflection a strain has been set up which is as great as the stress causing rupture in a specimen of the material, the height or vertical length of which specimen does not materially exceed its smallest diameter. Without entering upon the question of the magnitude of this excess, which has been variously estimated, it may be safely placed at that point where the strain generated by the element of bending becomes inappreciable, and it is probable that endwise compression is always accompanied with bending strains. The questions examined, however, in this paper are mainly, what is the maximum strain in pillars compressed endwise in the center of resistance by weights less than the breaking weights, or by weights not applied in the center of resistance.

Experiment shows that where a given pillar is deflected under increasing loads, the deflections increase at a greater ratio than the loads causing them, and the reverse is also true. The less the compressing force, the greater the ratio of reduction in the deflection.

The maximum strain at the middle of the intrados of a pillar bent to the breaking point may be assumed to be C, the ultimate resistance of the material to crushing. Let W be the breaking load of a solid 1-in. square pillar; the moment of resistance to bending is as follows :

If the vertical strain of the breaking load is deducted from the maximum strain C, the quantity $C - W$ remains to resist the bending moment $\delta\ W$, where δ is the breaking deflection and $\frac{d^3}{6}(C - W) = \delta\ W$, and as $d = 1$ the total strain $C = W(1 + 6\ \delta)$.

Now, let S be a strain $S < C$ and $W_1 < W$ and $\delta_1 < \delta$, then $S = W_1 \times (1 + 6\ \delta_1)$. Compare these two equations, and it appears that the relation of S to C can be as the respective weights W_1 to W only on the assumption that $1 + 6\ \delta_1 = 1 + 6\ \delta$, or that $\delta_1 = \delta$. It seems clear, therefore, that safe loads of pillars are not proportional to the breaking weights, but must be referred to a permissible maximum strain.

In building, the practice still prevails of considering cast-iron pillars planed top and bottom as subject to compound flexure as perceived in the testing machine. That this is not the case needs no special argument here, as the fact is universally recognized.

Breaking loads are computed by Gordon's and other formulas based upon Hodgkinson's experiments. In these experiments endwise compression was applied to the specimen as nearly as practicable in the center of resistance, or at least in the center of gravity of the pillar tested, which is also the center of resistance, unless the material is heterogeneous.

On pages 382 and 383 of this paper is an analysis of the bending strains caused by eccentric loads. The exact relation of the eccentric breaking load to the concentric breaking load is expressed in a formula.

Endwise Compression.

A specimen of cast-iron 1 in. square is extended 0.0017 of its length by a force of about 16 000 lbs. applied at the ends, and when so extended it is torn asunder. If the specimen is 5 ins. long, its extension

under a tensile strain of 16 000 lbs. is $5 \times 0.0017 = 0.0085$; when 10 ins. long, it extends under the same strain $10 \times 0.0017 = 0.017$; and when 100 ins. long, it is extended $100 \times 0.0017 = 0.17$ in., but in each event it requires a strain of neither more nor less than 16 000 lbs. to cause rupture.

A short specimen of cast-iron of the same sectional area of 1 in. is crushed by a compressing force of about 96 000 lbs.; but as the specimen becomes longer, the force required to disintegrate it becomes rapidly less. Thus, when the specimen is 5 ins. long, it will break under 85 000 lbs.; when 10 ins. long, under 45 000 lbs.; when 15 ins. long, under 30 000 lbs.; when 20 ins. long, under 23 000 lbs.; when 40 ins. long, under something less than 8 000 lbs.; and when 80 ins. long, under 2 000 lbs.

Now the main difference in the behavior of two specimens acted upon by compression and tension is that the latter specimen remains in a straight line during the application of force, while the former becomes curved. It is said, therefore, that a pillar or strut bends or deflects under endwise compression. Unequal compression, though not a primary cause, is a result of the process of bending. This involves a brief consideration of the nature of elasticity.

Elasticity is the internal force which in matter persists in maintaining atomic or molecular relations. The sum of persistence which a given kind of matter is capable of constitutes its potential elasticity. External force applied to given matter brings into activity an equal amount of elastic force, and while this external force remains in action, the potential energy of elasticity of the matter in question is reduced by that amount. When the external force is equal to the internal potential resistance of elasticity, the specimen is on the point of disruption. Any additional amount of external force, no matter how small, will cause disruption.

The exertion of the elastic force inherent in matter is always accompanied with fatigue. This means that the potential energy of elastic force is reduced by exertion. The amount of fatigue caused by comparatively small increments of external force (as compared in magnitude with the total potential energy of the elastic force of a given material) is imperceptible to the senses unless scrutinized by special sensitive instruments of measurement. The extent to which this is the case is known as the elastic limit of the material, and the amount

of fatigue which becomes perceptible beyond the elastic limit is called the permanent set. Within the elastic limit the increments of compression and extension are nearly the same for every equal amount of additional external force applied, so much for every additional 1 000 lbs. This is called uniform elasticity. Beyond the elastic limit, compression and extension increase with every additional equal amount of external force applied. This is called variable elasticity.

Within the elastic limit each pound of force exerted results in an equal amount of compression, or an equal amount of extension, though the amounts of compression and extension are never the same in any one material. In matter uniformly elastic, the number of pounds of external force divided by the resulting compression is a constant, or, if divided by the resulting extension, the quotient will be another constant. These constants are called the moduli of elasticity of the given material. So each material has its special moduli of elasticity for compression and for extension, and if the modulus of elasticity is expressed by E, the weight by W, the amount of compression by C, and that of extension by T, the modulus of elasticity for compression by E_c and that for tension by E_t, it follows:

$$\frac{W}{C} = E_c \quad \text{and} \quad \frac{W}{T} = E_t$$

$$W = E_c\, C \quad \text{"} \quad W = E_t\, T$$

$$\frac{W}{E_c} = C \quad \text{"} \quad \frac{W}{E_t} = T$$

Within the range of variable elasticity each number of pounds of strain has its special modulus of elasticity; hence, the convenience of its use in analysis is lost, and stress must be referred directly to the amount of the compression or extension caused by it in matter.

The compression per square inch area caused in cast-iron by weights ranging from 1 000 to 96 000 lbs., and those of wrought iron from 1 000 to 52 000 lbs., as presented in Tables Nos. 1 and 2, are derived from curves based upon the results of Hodgkinson's experiments.

Table No. 1 gives in column 2 the compression of 1 in. area of cast-iron for weights from 96 000 to 1 000 lbs., also in column 3 the compression due to the last 1 000 lbs. applied. It will be seen from the last that the elasticity of compression of cast-iron is variable throughout.

Table No. 2 gives in column 2 the compression of 1 in. area of wrought iron for loads given in column 1; also in column 3 the compression due to the last 1 000 lbs.

It will be seen that for wrought-iron pillars the additional increment of compression for every additional 1 000 lbs. is 0.00004 up to 12 000 lbs., and 0.00005 from 12 000 to 24 000 lbs. Twenty-four thousand to 28 000 lbs., therefore, is accepted as the limit of elasticity of wrought iron. The average increase per 1 000 lbs. up to 24 000 lbs. is 0.000045; therefore, 1 000 ÷ 0.000045 gives the modulus of elasticity for the compression of wrought iron as 22 000 000.

For the extension of wrought iron within the elastic limit the modulus is found by experiment to vary between 25 000 000 and 28 000 000.

Deflection.

For greater convenience of analysis and of arriving at tables which give the breaking and safe weights per inch area for lengths expressed in terms of diameters or radii of gyration, let the diameter of the pillars be $d = 1$; let them be solid and square in form; hence, the sectional area will also be $a = 1$; the moment of inertia will be $\frac{1}{12}$; the moment of resistance $\frac{1}{6}$, and the pillar length expressed in diameters will represent so many inches.

If a pillar is compressed vertically by its breaking weight and is in some way prevented from bending, its potential elastic energy for compression still remaining is $C - W$, where C is the crushing weight of the material, and the potential energy of tension still remaining is $T + W$ when T represents the ultimate resistance of the material to tension.*

If, now, the pillar is left free to bend, then at the breaking point this remaining potential energy is exhausted; hence, the bending moment $\delta\ W$, δ being the deflection, is in equilibrium with the potential energy multiplied by the moment of resistance; hence,

$$\delta\ W = \frac{1}{6}(C - W) = \frac{1}{6}(T + W) \text{ and } \delta = \frac{C - W}{6\ W} = \frac{T + W}{6\ W}$$

* When a body is compressed or extended by a force less than its ultimate resistance to compression or tension, the balance of compression or extension possible may be considered to be its potential energy for resisting another external force.

Let a body capable of an ultimate resistance to compression C, or of resistance to extension T, be compressed in the one instance by a force W, or extended by a force W, in either event its potential energy will be $C - W$ or $T - W$. But if the external force is one of compression, in either case the potential energy of resistance to compression is $C - W$, and that to extension is $C + W$.

Under conditions of uniform elasticity this value of δ is absolutely correct. But inasmuch as elasticity is variable, it would be true only if the horizontal sectional planes drawn transversely to the pillar length were to become curved during the process of bending.

To ascertain exact deflections related to certain breaking loads from an assumed condition of compression, or of compression and tension at the middle of a pillar bent to the breaking point, the following process is practicable:

Let *a b c d* (Fig. 1) represent a diagram of compression at the middle of a square cast-iron pillar 1 in. on a side bent to the breaking point. The stress at *b*, the intrados of the pillar, is assumed to be 96 000 lbs., the crushing stress of the material. The compression due to a stress of 96 000 lbs., as given in the table, is 0.026. At the point *a*, the extrados of the bent pillar, the compressive strain is assumed to be 32 000 lbs. Compression at the intervening points between *a* and *b* is as marked upon the vertical lines drawn at those points, and the strain due to these respective compressions, as given in Table No. 1, is the number of pounds marked at the bottom of the vertical lines. *A B C D* (Fig. 1) represents a strain diagram at the middle of the pillar divided into ten laminas representing the mean stress of each lamina. This diagram gives by computation an area of 71 080 lbs. and a center of gravity 0.0692 distant from the axis of the pillar, which distance constitutes the deflection.

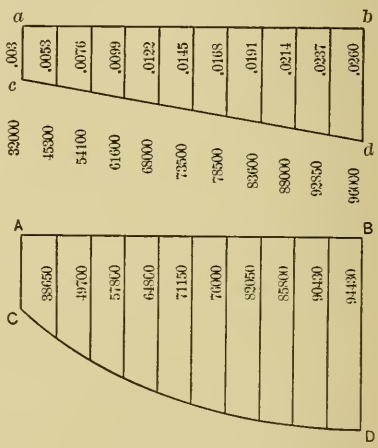

Fig 1.

A series of diagrams like these with varying strains at *a* forms a basis for tables giving the breaking weight, deflection, and, as will be shown hereafter, the breaking radius and breaking length of all possible pillars.* Tables Nos. 3 and 4 have been prepared in this

* This process of ascertaining the breaking weights, deflections, radii and corresponding breaking lengths, as given in Tables Nos. 3 and 4, is absolutely correct. The method first stated (page 123) of computing deflection by the formula $\delta = \frac{C - W}{6\ W}$ is less accurate and so is the resulting determination of the radius, and consequently of the pillar length. The result is that for given pillar lengths the breaking weights are less than those based upon

manner; Table No. 3 is for cast-iron pillars, and Table No. 4 for wrought-iron pillars.

RADIUS.

Let $a\ b\ c\ d\ g$ (Fig. 2) represent the strain diagram at the middle of a pillar bent to the breaking point under a breaking load $W\ a\ b = T$ (tension) and $d\ g = C$ (the ultimate crushing resistance of the material). Then $m\ b\ o\ n\ g$ represents the total resistance to bending, and $a\ d\ n\ m$ the resistance to vertical compression.

If the latter is assumed to pervade the whole pillar, the former determines the varying positive and negative strains and their accompanying compression and extension, which constitute the elements that determine the radius.

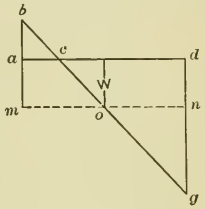

FIG. 2.

At the intrados, the pillar always shortens in bending. At the extrados, it may shorten less, not at all, or it may lengthen by tension. The shortening at the breaking point at the middle of the pillar is $d\ g = c$. That part of $d\ g$ which represents resistance to bending ($n\ g$) becomes continually less at points approaching the top and bottom of pillar, and is nil at the top and bottom.

The average of the strains in point (if the bending curve of the pillar is assumed to be the same as that of a rectangular beam under a transverse load in its center) is $p\ (C - W) = 0.6125 \times (C - W)$. A circular curve drawn through the top, the bottom, and the middle of the intrados of a pillar at the breaking point therefore represents a uniform strain of compression of $W + p\ (C - W)$, and a similar circular curve at the extrados represents a uniform strain of

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the assumption of a curved resistance diagram at the middle of the pillar at the point of breaking. Exactly how much this difference amounts to may be ascertained by comparing Tables Nos. 1 and 2 with Tables Nos. 3 and 4.

Figs. 3 and 4 show the curves of breaking weights for respective pillar lengths in accordance with Tables Nos. 1 and 3 and Nos. 2 and 4 for more convenient comparison. But inasmuch as breaking weights are of themselves no criterion of safe loads, the formula $\delta = \frac{C - W}{6\ W}$ has been used in the following analysis of deflections under loads less than the breaking weight with results that show no material difference when applied to either of Tables Nos. 1 and 3 or Nos. 2 and 4, except in the case of long wrought-iron pillars, which show somewhat increased safe loads when computed in accordance with Table No. 4.

Inasmuch as the breaking loads for both cast and wrought iron, as given in Tables Nos. 1 and 2, more nearly coincide with the results of Hodgkinson's experiments, and inasmuch as a moderate reduction of the safe loads of long columns secures reasonably moderate deflections, it has been deemed best to compute the tables of safe loads in accordance with Tables Nos. 1 and 2.

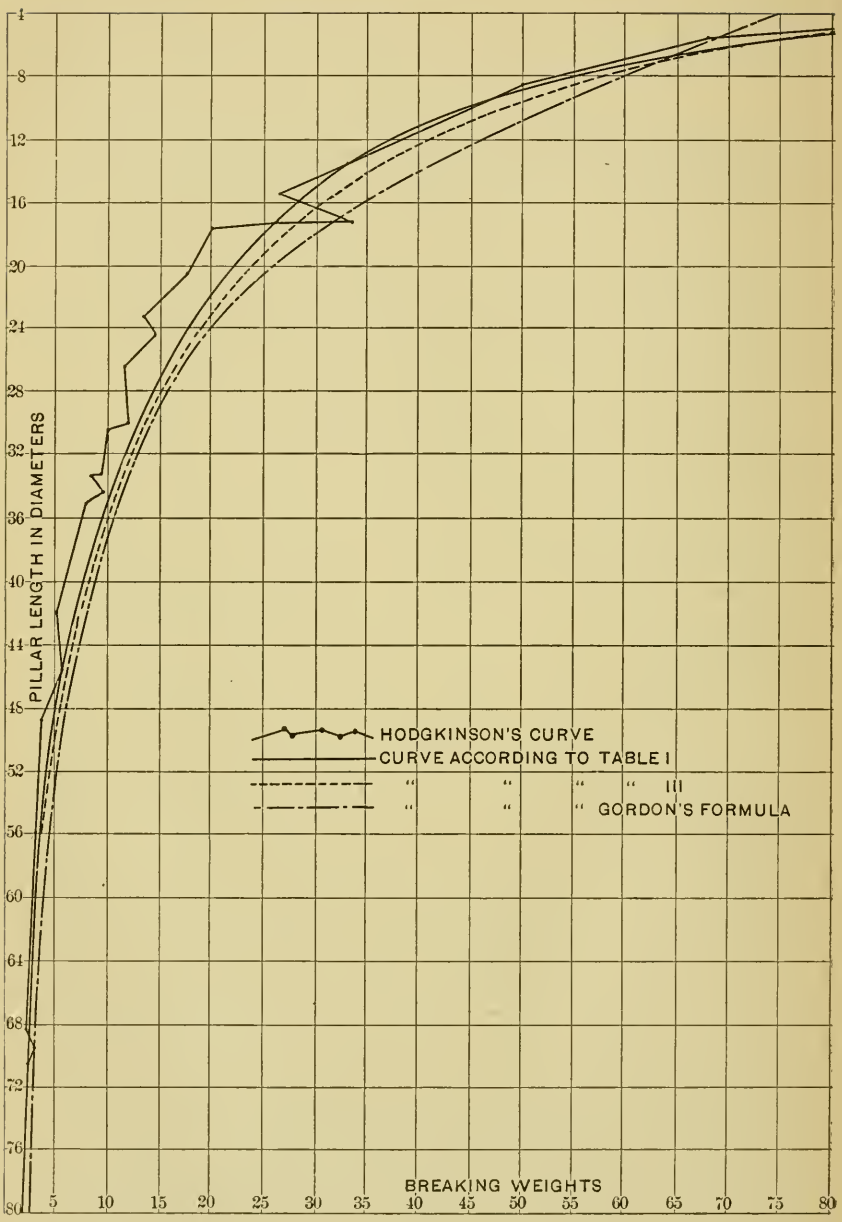
PILLAR LENGTH IN DIAMETERS
4
8
12
16
20
24
28
32
36
40
44
48
52
56
60
64
68
72
76
80
BREAKING WEIGHTS
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
HODGKINSON'S CURVE
CURVE ACCORDING TO TABLE I
" " " " III
" " " GORDON'S FORMULA

FIG. 3.

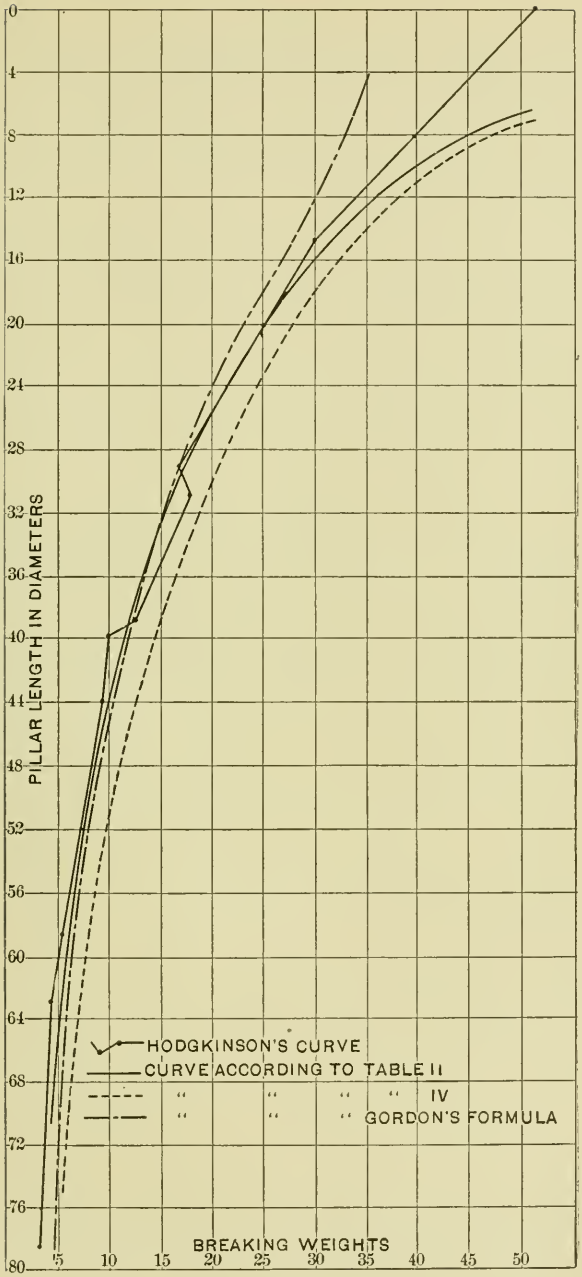
PILLAR LENGTH IN DIAMETERS
0
4
8
12
16
20
24
28
32
36
40
44
48
52
56
60
64
68
72
76
80
BREAKING WEIGHTS
5
10
15
20
25
30
35
40
45
50
HODGKINSON'S CURVE
CURVE ACCORDING TO TABLE II
" " " " IV
" " " GORDON'S FORMULA

FIG. 4.

$W - p\ (C - W)$ which may be negative or positive, a strain of compression or tension. The radius and deflection of a pillar are the sole elements of its length (as will be shown hereafter), and though the circular curve which passes through the middle and top and bottom differs from the bending curve, this difference of length may be neglected without appreciable error.

The radius of a circular curve is the radius of any portion of it, being dependent upon the compression at the intrados, and the compression or extension at the extrados. If compression be expressed by the sign C_p and extension by the sign E_x and $a\ b\ c\ d$,

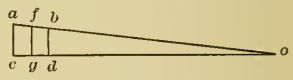

FIG. 5.

Fig. 5 represents a portion of the pillar in a circular curve, of which $f\ g$ is the axis, then $o\ g$ is the radius R, and if the diameter of the pillar is d, then $a\ c;\ b\ d = R + \frac{d}{2} = R - \frac{d}{2}$,

$$\text{and if } d = 1 \text{ then } R = \frac{a\ c + b\ d}{2\ (a\ c - b\ d)}.$$

Let the height of $a\ b\ c\ d$ be considered as 1 prior to deformation by bending, then $a\ c = 1 - C_p\ [W - p\ (C - W)]$, and

$$b\ d = 1 - C_p\ [W + p\ (C - W)] \text{ and}$$

$$R = \frac{2 - [C_p\ [W - p\ (C - W)] + C_p\ [W + p\ (C - W)]]}{2\ [C_p\ [W + p\ (C - W)] - C_p\ [W - p\ (C - W)]]}$$

When $p\ (C - W)$ is greater than W, then $W - p\ (C - W)$ becomes negative and denotes tension.

Example.—To find the radius for a breaking weight $W = 48\ 000$ lbs., the ultimate resistance of the material to crushing being $C = 96\ 000$ lbs. $C - W = 48\ 000$ and $p\ (C - W) = 29\ 400$; $W + p\ (C - W) = 48\ 000 + 29\ 400 = 77\ 400$ lbs., $W - p\ (C - W) = 18\ 600$ lbs. By Table No. 1 the compression for 77 000 lbs. is 0.016045. For the additional 400 lbs. at the rate of the last 1 000 lbs., which is 0.000454; it is 0.000182; therefore, for 77 400 lbs. the compression is $0.016045 + 0.000182 = 0.016227$. The compression due to 18 600 lbs., the uniform stress at the extrados by Table No. 1, is 0.001606; hence,

$$R = \frac{2 - (0.001606 + 0.016227)}{2\ (0.016227 - 0.001606)} = \frac{2 - 0.017833}{2\ (0.014621)} = 67.7.$$

Table No. 1 gives the uniform strains for cast-iron at the intrados in column 6 and those of the extrados in column 7. Column 4 of the same table gives the strain at the middle of the extrados. It will be observed that this strain is one of compression throughout the pillar

area when the breaking weights range from 96 000 lbs. down to 48 000 $= \frac{1}{2} C$; at 48 000 lbs. the stress in the middle of the pillar at the extrados is nil. From 48 000 down to 40 000 lbs. breaking weight, the strains at the middle at the extrados are tensional, beginning with 0 and ranging up to 16 000 lbs., the assumed ultimate resistance of the material to tension. At the intrados in the middle of the pillar down to a breaking load of 40 000 lbs. the strain is constant at the ultimate resistance of the material to crushing, which is assumed at 96 000 lbs. When cast-iron pillars exceed a length of 11.3 diameters, they break under a load less than 40 000 lbs. by rupture at the extrados; hence, the strain at the middle of the intrados becomes less than 96 000 lbs., and, as is shown by Table No. 1, when a 1-in. square cast-iron pillar is 115.1 ins. long, it breaks by tension at the extrados, while at the middle of the intrados the strain of compression has run down to 18 000 lbs.

It may be noticed also that deflection becomes greater when the pillar becomes longer and the breaking weight becomes less. As the breaking weights descend from 96 000 to 1 000 lbs., deflection begins with 0.00175 and ends with 2.83333. The radius, on the other hand, is largest with the greatest breaking weight and becomes less when the breaking weight becomes less. It is least at a breaking weight of 48 000 lbs. From this point downward in the scale of breaking weights the radius becomes larger again.

Both radius and deflection depend upon the resistance of the pillar to bending at the breaking point. This resistance is the potential energy of the material still remaining after the vertical force of endwise compression is deducted from the ultimate strength of the material; $C - W$ or $T + W$ are the measure of this potential energy. The greater the elastic force of $C - W$ or $T + W$, the greater the deflection.

This accounts for the fact that pillars break with smaller deflections than beams under a transverse load. In the latter, resistance to bending is the total ultimate strength of the material, while in the former a portion of this strength is consumed by endwise vertical compression.

Length of a Pillar.

Let $a\,b$ (Fig. 6) represent a small portion of a circular curve, the radius of which is $a\,o$, and its diameter $a\,d$; then the curve $a\,b$ may without serious error be considered as a straight line, and the triangle

$a\,c\,b$ is homologous to the triangle $a\,b\,d$ and $a\,c : a\,b = ab;\ b\,d$ and $a\,c \times b\,d = a\,b^2$.

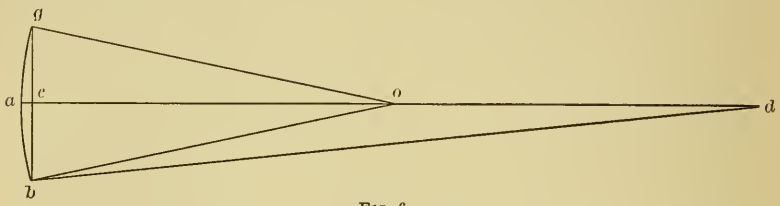

FIG. 6.

Let $b\,g$ represent the length of a pillar bent with a deflection $\delta = a\,c$ and $b\,g = L$, hence $L = 2\,a\,b$ and $a\,b^2 = \frac{L^2}{4}$ and $b\,d = 2\,R$; then by substituting these values of $a\,c$, $b\,d$ and $a\,b$ in $a\,c \times b\,d = a\,b^2$.

$$2\,R\,\delta = \frac{L^2}{4} \text{ and } L^2 = 8\,R\,\delta \text{ and } L = \sqrt{8\,R\,\delta}.$$

Tables Nos. 1, 2, 3 and 4 give the length of pillars corresponding to the respective breaking weights for solid square cast and wrought-iron pillars, on the assumption of straight and curved diagrams of resistance at the middle of the pillar. Fig. 3 gives resistance curves at the breaking point of solid square cast-iron pillars of all lengths and breaking weights in accordance with Tables Nos. 1 and 3, also with Gordon's formula and Hodgkinson's experimental results. Fig. 4 gives the same for solid square wrought-iron pillars.

Eccentric Loads.

The foregoing analysis refers to pillars compressed endwise by a force acting in the center of gravity of the resistance, which in a homogeneous pillar is also the center of gravity of its cross-section. In that case stress is assumed to be equally distributed at the top of the pillar, also at the bottom.

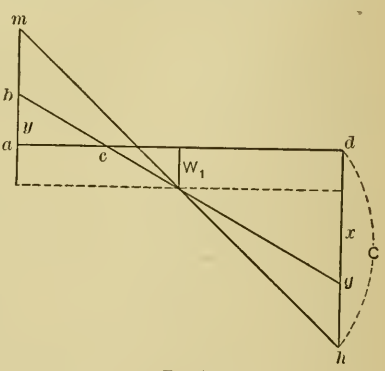

FIG. 7.

When the force compressing the pillar is not acting in its center, and the eccentricity is n (a fraction of its diameter), then the distribution of stress at the top of the pillar is not uniform, but may be expressed by the diagram, Fig. 7, where W_1 being 1, x denotes strain

of compression at the intrados x, 1 or $x\ W$, and y the strain at the extrados, y, 1 or $y\ W_1$; the latter may be a strain of compression or tension or nil, and

$$\frac{x-1}{6} = \frac{y+1}{6} = n, \text{ and}$$

$$x = 6\,n + 1 \text{ and } y = 6\,n - 1.$$

At the point of breaking, the strain at the intrados in the middle of the pillar has increased to C, the crushing strain of the material, hence the moment due to deflection δ_1 is resisted by $\frac{g\ h}{6}$. The resistance to the total bending is therefore

$$\frac{C - W_1}{6} = (n + \delta_1)\ W_1$$

Let $a\ b\ c\ g$ (Fig. 8) represent the upper half of the pillar bent by the load W_1, placed upon the top of the pillar at the distance n from its center of resistance (where n represents a fraction of the diameter of the pillar), and let δ be the resultant deflection and $\frac{C - W_1}{6}$, the resistance to bending

$$\frac{C - W_1}{6} = (n + \delta_1)\ W_1 \ldots\ldots (1)$$

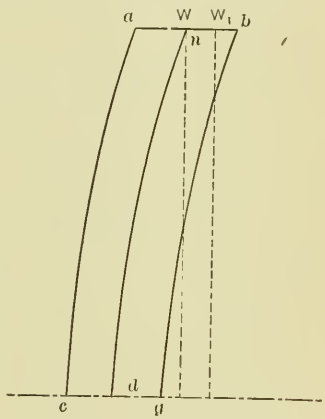

FIG. 8.

Let W, the breaking weight of the pillar in question, when acting in the center of resistance, be placed at the center, the moment of resistance to bending will be $\frac{C - W}{6}$ and the pillar will bend with a deflection $\delta < \delta_1$ and $\delta\ W : \delta_1\ W = C - W : C - W_1$ or $\delta : \delta_1 = C - W : C - W_1$ and $\delta_1 = \delta\ \frac{C - W_1}{C - W}$, but as $\delta = \frac{C - W}{6\ W}$, then $\delta_1 = \frac{C - W_1}{6\ W}$.

When this value of δ_1 is introduced in equation (1), then

$$W_1 = \pm \sqrt{\left(\frac{C + W + 6\,n\ W}{2}\right)^2 - C\ W} + \frac{C + W + 6\,n\ W}{2}$$

In the case of the cast-iron pillars, the bending resistance is $\frac{T + W}{6}$, then

$$W_1 = \pm \sqrt{\left(\frac{W - 6\,n\ W - T}{2}\right)^2 + T\ W} + \frac{W - 6\,n\ W - T}{2}.$$

Table No. 5 gives the breaking weights for cast-iron pillars of length, in diameters varying from 5 to 50, when compressed by loads placed to one side of their neutral axis, for eccentricities of 0.1, 0.3, 0.5, 0.7 and 1.0. Table No. 6 gives breaking loads for wrought-iron pillars for lengths in diameters from 7 to 44 for the same eccentric loads.

The Deflection and Radius Caused by $W_1 < W$ when Placed in the Center of Resistance.

Let a pillar of the length L, breaking deflection δ, breaking weight W, and breaking radius R, be bent to a deflection $\delta_1 = \frac{\delta}{2}$ or $\delta_{11} = \frac{\delta}{4}$; the respective weights will be less than W, say $W - a$, and $W - (a + b)$ and the corresponding bending moments are $\delta_1 (W - a)$ and $\delta_{11} (W - a - b)$ and the bending radii R_1 and R_{11}. Inasmuch, however, as L is constant, then $L^2 = 8 \delta R = 8 \delta_1 R_1 = 8 \delta_{11} R_{11}$ or $\delta R = \delta_1 R_1 = \delta_{11} R_{11} = \ldots \delta_n R_n$. The value of $R R_1 R_{11}$ is always a function of the respective deflections, δ, δ_1 and δ_{11}, irrespective of bending weight.

The bending moment, on the other hand, depends on the bending weight as well as on the deflection. The greatest bending strain occurs at the middle of the intrados of the pillar, and at the breaking point it is C, the crushing resistance of the material when deflected to the point δ. With a deflection δ_1, the greatest stress at the middle of the intrados will be $S < C$, and with a deflection δ_{11} it is $S_{11} < S$.

Comparing the bending moments with the respective resistance, $\frac{C - W}{6} = \delta W$; $\frac{S - W_1}{6} = \delta_1 W_1 = \delta_1 (W - a)$ and

$$\frac{S_{11} - (W - a - b)}{6} = \delta_{11} (W - a - b) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

If elasticity is assumed to be uniform, and $S - W$ is the internal stress which resists bending and the compression due to $S - W$ is $\frac{S - W}{E}$, E being the modulus of elasticity of the material in question, the radius may be found by comparing the length of any unit of measure at the intrados with a corresponding unit of the pillar axis, without regard to the vertical compression which is uniformly distributed. In that case compression at the pillar axis is nil and at the intrados it is $\frac{S - W}{E}$ as stated previously, and $1 - \frac{(S - W_1)}{E} : 1 = R_1 - \frac{d}{2} : R_1$

and the radius $R_1 = \frac{1E}{2p (S - W_1)}$, hence

$$R_1 = \frac{E}{2p\,(S - (W - a))} \text{ and } R_{11} = \frac{E}{2p\,(S_{11} - (W - a - 6))} \text{ and}$$

$$R_1 : R_{11} = S_{11} - (W - a - b) : S_1 - (W - a) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

From equation (2) :

$$S = (W - a)\,(1 + 6\,\delta_1) \text{ and } S_{11} = (W - a - b)\,(1 + 6\,\delta_{11})$$

and as $\delta_1 = \frac{C - W}{12\,W}$ and $\delta_{11} = \frac{C - W}{24\,W}$

$$S = (W - a)\left(1 + \frac{C - W}{2\,W}\right) \text{ and}$$

$$S_{11} = (W - a - b)\left(1 + \frac{C - W}{4\,W}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Substitute values of S and S_{11} in equation 3.

$$R_1 : R_{11} = (W - a - b)\left(1 + \frac{(C - W)}{4\,W}\right) - (W - a - b):$$

$$(W - a)\left(1 + \frac{C - W}{2\,W}\right) - (W - a)$$

$$R_1 : R_{11} = (W - a - b)\,\frac{(C - W)}{4\,W} : (W - a)\,\frac{(C - W)}{2\,W}$$

$$R_1 : R_{11} = \frac{(W - a - b)}{2} : W - a$$

$$R_{11} = 2\,R_1\,\frac{W - a}{(W - a - b)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

But inasmuch as $R_{11}\,\delta_{11} = R_1\,\delta_1$ and $\delta_1 = 2\,\delta_{11}$, hence $2\,R_1$ must be $= R_{11}$ in all cases. Therefore equation (5) is true only when a and b are nil.

At all points short of the breaking deflection it is known by experiment that there is no equilibrium during the process of bending under a breaking weight until the breaking deflection is reached, and that resistance to the bending moment is least at the smallest deflection, and becomes greater as the breaking deflection is approached.

Yet at half the breaking deflection $\delta_1 = \frac{\delta}{2}$

$$\delta_1\,W = \frac{S - W}{6};$$

at the breaking point $\delta\,W = \frac{C - W}{6}$; hence,

$$S - W : C - W = \delta_1\,W : \delta\,W = 1 : 2, \text{ and}$$

$$S - W = \frac{C - W}{2}.$$

If $S - W$ represented an internal force of resistance to the bending moment, then experiment would show a condition of equilibrium ; but this not being the case $\frac{S - W}{6}$ only partially consists of compression, a compression due to a strain less in magnitude than $S - W$, in fact a stress equal to $(W - a)$.

But $S - W$ is the true measure of the radius, hence the radius is the result of two forces. The one is $\delta_1 (W - a)$ which is the force of compression at the intrados in excess of that at the pillar axis, and the force $\delta_1 a$, which for the present may be attributed to the vertical stress of a weight $W - a$ without a definition of its exact nature, simply because endwise compression is the sole cause of any and all deformation of the pillar. An example will illustrate the foregoing more clearly:

Given a solid 1-in. square cast-iron pillar and 9.5 ins. long, then $C = 96\ 000$ lbs., $W = 48\ 000$ lbs., $\delta = 0.16666$, $R = 67.7$ and $\delta R = 11.3$; consequently any $\delta_1 R_1$ must also be equal to 11.3, and if $\delta_1 = \frac{\delta}{2} = 0.08333$, then $R_1 = 135.4$.

The value of W_1 being unknown further than that it is less than W, it must be something between 48 000 lbs. and nil. Whatever its magnitude; when $\delta_1 = \frac{\delta}{2}$, the radius R_1 must be 135.4.

Let W be 48 000 lbs., then $\delta_1 W = \frac{\delta W}{2}$ and as $\delta W = \frac{C - W}{6}$ then $\frac{\delta_1}{2} W = \frac{C - W}{12} = \frac{S - W}{6}$ where S is the total strain at the intrados when the deflection is $\delta_1 = 0.08333$, and $\delta_1 W = \frac{S - W}{6} = 4\ 000$ lbs., and $S - W = 24\ 000$ lbs.; hence $S = 72\ 000$ lbs. S being known, the radius may be computed by the formula developed on page 380, by substituting S for C and

$$R_1 = \frac{2 - [C_p (W - p (S - W)) + C_p (W + p (S - W))]}{2 [C_p (W + p (S - W)) - C_p (W - p (S - W))]} = 140.3.$$

If $W = 44\ 000$ lbs., $S = 66\ 000$ lbs. and $R_1 = 172.7$.

If $W = 40\ 000$ lbs., then $S = 60\ 000$ lbs. and $R_1 = 219.5$.

If $W = 39\ 070$ lbs. (which is, as will appear hereafter, to be the true weight which bends the pillar with a deflection $\delta = 0.08333$), then $S - W = 19\ 534$ and $S = 58\ 606$ and $R_1 = 233.1$.

Table No. 7a gives the values of S, $(S - W)$ and R for possible W_1 from 48 000 down to 24 000 lbs.

Inasmuch as the radius corresponding with a deflection $\delta_1 = 0.08333$

cannot be more than 135.4, it appears that whatever the bending weight which results in a resistance of $\frac{S - W_1}{6}$ the radius to correspond with that resistance is less than the radii shown in Table No. 7A, whence $S - W_1$ must be correspondingly greater than those appearing in the table.

Whatever the value of $S - W_1$, or X as it will be called hereafter, it is greater than the bending moment.

It constitutes the resistance to a force acting transversely to the pillar length to the extent of the deflection. It appears that when a pillar bends under any weight W or $W_1 < W$ the deflection is not owing primarily to diversity of compression, but to a sliding motion of the particles in a horizontal direction which results in a displacement of the compressing force to one side of the pillar axis, hence unequal compression of the transverse area of the pillar.

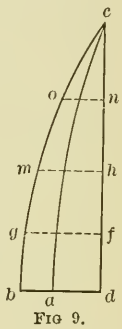

FIG 9.

In Fig. 9 the curve of deflection $a\ c$ is projected, which is due to unequal compression, and $b\ c$ is the curve of total deflection. This means that $b\ d = \delta_1$ (the actual deflection), and $a\ d$ is that part of the actual deflection which is due to diversity of compression, and it means further that during the process of bending the point d has moved horizontally to b while c remained stationary, also that the points of f, h and n have moved respectively to g, n and o.

If the pillar is assumed to be compressed in a testing machine, c is stationary and d, f, h and n are movable. This motion doubtless begins with any pressure, no matter how small, and when W_1 is very small, $b\ d$ also will be proportionately small. Yet as the line of compression continues to be a straight line between the poles of the pillar axis, and as the pillar axis in the meantime has become a curve, unequal compression is the result. It will be shown hereafter that when the compressing force is very small as compared with the breaking weight, $a\ d$ is also very small as compared with $b\ d$ and when the compressing force becomes the breaking weight, $a\ b = b\ d$.

Resistance to Bending, for $W_1 < W$.

At the breaking point the amount of bending depends upon the bending weight and upon the strength of the material; this means the total potential energy of the material still remaining after a portion of

it has been consumed in resisting vertical stress. The formula $\delta = \frac{C - W}{6\,W}$ means that a pillar bends at the breaking point as much as the resistance of the material will permit and inversely as the bending force. Deflection at the breaking point is an element in determining the length of the pillar, and deflection is as the square of the pillar length limited by the radius, which is also as the square of the pillar length limited by the deflection. Elasticity does not enter into determining the breaking deflection. But when it is intended to compare bending and resistance to bending for weights less than the breaking weight, elasticity becomes an element in comparing the condition of it for a weight $W_1 < W$ with a corresponding condition at the breaking point.

If resistance to bending under the weight $W_1 < W$ be called X then

$$X : C - W = W_1\,E : W\,E_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Now the radius expressed in terms of elasticity (E) is

$$R = \frac{E}{2\,p\,(C - W)}$$

$$\text{Deflection } \delta = \frac{C - W}{6\,W}$$

The pillar length is $L^2 = 8\,R\,\delta$; hence

$$L^2 = \frac{8\,E}{12\,p\,W} = \frac{2\,E}{3\,p\,W} \text{ and } L_1{}^2 = \frac{2\,E_1}{3\,p\,W_1}, \text{ and}$$

$$L^2 : L_1{}^2 = \frac{E}{W} : \frac{E_1}{W_1} = E\,W_1 : E_1\,W \ldots\ldots\ldots\ldots (7)$$

Comparing equations (6) and (7), $X : C - W = L^2 : L_1{}^2$ where L is the pillar length, and L_1 the length of a pillar of which W_1 is the breaking weight.

Tables Nos. 7 A and 7 B give the X as computed by the above formula for various weights less than the breaking weight, also the respective radii and deflections deduced from the value.

It will be seen that in all cases, whatever the value of W, that $\delta\,R = \delta_1\,R_1 = \delta_{11}\,R_{11} = \ldots\ldots\ldots\ldots\ldots\ldots\

To Find the Radius.—The radius R may be found by the formula—

$$R = \frac{2 - [C_p (W + p (C - W)) + C_p (W - p (C - W))]}{2 [C_p (W + p (C - W)) - C_p (W - p (C - W))]}$$

by substituting $W_1 = 24\ 000$ lbs. for W and $7\ 050 = p X$ for $p (C - W)$.

$$R_1 = \frac{2 - (C_p (24\ 000 + 7\ 050) + C_p (24\ 000 - 7\ 050)}{2 (C_p (24\ 000 - 7\ 050) - C_p (24\ 000 - 7\ 050)}$$

$$R_1 = \frac{2 - (C_p\ 16\ 950 + C_p\ 31\ 050)}{2 (C_p\ 31\ 050 - C_p\ 16\ 950)}$$

$$R_1 = \frac{2 - (0.002897 + 0.001451)}{2 (0.002897 - 0.001451)} = 690.1.$$

To Find the Deflection.—The breaking radius of a pillar 9.5 ins. in length as previously described is $R = 67.7$, and the breaking deflection $\delta = 0.16666$, hence $\delta\ R = 11.3$ and $\frac{R\delta}{R_1} = \frac{11.3}{690.1} = 0.01637$, the deflection due to the weight $W_1 = 24\ 000$ lbs.

The deflection may be computed however from the quantity pX without reference to the radius by the formula $\delta_1 : \delta = C_1 : C$, where C_1 is the difference of compression between that of the intrados and extrados, and C is the compression of $W + p (C - W) - (W - p (C - W))$, hence

$$\delta_1 = \frac{\delta\ C_p (W_1 + pX) + C_p (W_1 - pX)}{C_p (W + p (C - W)) - C_p (W - p (C - W))},$$

In this case $W + pX = 24\ 000 + 7\ 050 = 31\ 050.$

and $W_1 - pX = 24\ 000 - 7\ 050 = 16\ 950.$

By Table No. 1, compression of $31\ 050 = 0.002897$

" " $16\ 950 = 0.001450$

and the difference between them $= 0.001447$

By Table No. 1 may also be found

$$C_p (W + p (C - W)) - C_p (W - p (C - W)) =$$
$$0.016683 - 0.001606 = 0.014977$$

and as $\delta = \frac{1}{6}$, hence $\delta_1 = \frac{1 \times 0.001447}{6 \times 0.014977} = 0.0161$

The difference (0.00027) between this result and the previous one is owing to the fact that pillar lengths are developed only to the extent of one decimal, which is ordinarily quite sufficient for all practical purposes.

Table No. 7 A gives the values of $\frac{L^2}{L_1{}^2}$, of X and pX, the radii and

deflections of various weights less than the breaking weight of a solid square cast-iron pillar 9.5 in length.

Table No. 7 A shows the deflection of the

Breaking	weight,	$W = 48\ 000$ lbs.	to be	0.16666
"	"	$W_1 = 39\ 070$ lbs.	"	0.08333
"	"	$W_1 = 24\ 000$ lbs.	"	0.01655
"	"	$W_1 = 12\ 000$ lbs.	"	0.00528
"	"	$W_1 = 4\ 000$ lbs.	"	0.00155

Table No. 7 B gives the same data for a square solid wrought-iron pillar 19.6 diameters in length.

The deflection under loads less than the breaking load may also be computed directly from the resistance at the breaking point of the pillar $\frac{C - W}{6}$ as follows:

Resistance to bending under lesser weights than the breaking weight, as previously shown, is $X = (C - W) \frac{L^2}{L_1^2}$,

where L is the pillar length in question and L_1 is the breaking length of a pillar which breaks under W_1, the weight assumed as $W_1 < W$.

Let w be a force applied transversely upon the pillar at its middle, then $\frac{x}{6} = \frac{w\,L}{4}$ and $w = \frac{2\,X}{3\,L} = \frac{2}{3}\,\frac{(C - W)\,L^2}{L_1^2\,L} = \frac{2}{3}\,\frac{(C - W)\,L}{L_1^2}$ ..... (8)

If w is considered as a transverse load at the middle of the pillar, and the pillar is considered as a beam resting at the top and bottom, its deflection is $\delta_1 = \frac{w\,L^3}{48\,I\,E}$, and if the pillar as heretofore is solid and 1 in. square, I (mom. inertia) $= \frac{1}{12}$ and $\delta_1 = \frac{w\,L^3}{4\,E}$ ..... (9)

Substituting the value of w from equation (8)

$$\delta_1 = \frac{2}{3}\,\frac{(C - W)\,L^4}{4\,E\,L_1^2} = \frac{(C - W)\,L^4}{6\,E\,L_1^2}$$

Example.—Given the solid 1-in. square wrought-iron pillar 19.6 ins. in length, the breaking load of which is $W = 26\ 000$ lbs.; its breaking deflection $\delta = 0.16666$; resistance to crushing, $C = 52\ 000$ lbs.; hence, $C - W = 26\ 000$ lbs.; what is the deflection of this pillar when bending under a load of 12 000 lbs.?

The length of a pillar L_1 that breaks under 12 000 lbs. is 38.8 diameters, and $L_1^2 = 1\ 505.44$.

L being 19.6, hence $L^4 = 14757.9$. The average compression of wrought iron within the limit of elasticity is 0.000045 per 1 000 lbs., hence $E = 22\,220\,000$ and $\delta_1 = \frac{(C - W)\,L^4}{6\,E\,L_1^2} = \frac{26\,000 \times 14\,757.9}{6 \times 22\,220\,000 \times 1\,505.44} =$ 0.019. Table No. 7 B gives 0.01828.

Deflections of safe loads may also, within the limit of elasticity, be derived directly from the breaking deflection by the following method:

$$\text{If } \delta_1 = \frac{(C - W)\,L^4}{6\,E\,L_1^2} \text{ then } \delta = \frac{(C - W)\,L^4}{6\,E_1\,L^2} = \frac{(C - W)\,L^2}{6\,E_1} \text{ and}$$

$$\delta_1 : \delta = \frac{(C - W)\,L^4}{6\,E\,L_1^2} : \frac{(C - W)\,L^2}{6\,E_1} = \frac{L^2}{E} : \frac{L_1^2}{E_1}$$

Approximately the modulus of elasticity of the last 1 000 lbs. of W is to the modulus of elasticity of the last 1 000 lbs. of W_1 as $E : E_1$.

Fig. 10 explains the reason for this statement. Let $a\ b\ c\ d$ be the trapezoid which represents the resistance at the middle of the pillar to vertical load and to bending; then $f\,g$ is the bending force and the triangles $a\,m\,g$ and $n\,g\,d$ represent the resistance to bending, and $a\ m$ and $n\ d$ express a number of pounds, and m and n the mean of this number of pounds. Hence the average of the thousands of pounds located between d and a is to be found at n, and the modulus of elasticity of the 1 000 lbs. at n is also approximately the average of the moduli of the various thousands of pounds contained between d and a. This is approximately but not absolutely true, but the error involved is small and on the safe side.

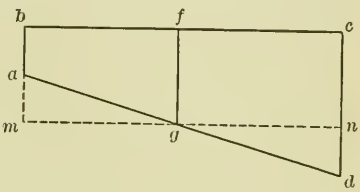

FIG. 10.

Now if the last 1 000 lbs. of $n = (W)\ 1\,000$, and of $m_1 = (W_1)\ 1\,000$, then $E = \frac{(W)\ 1\,000}{C_p\ 1\,000}$ and $E_1 = \frac{W_1\ 1\,000}{C_p\ 1\,000}$ and $\delta_1 : \delta = L^2\,(C_p\,W_1) \cdot \frac{1\,000}{C_p\ 1\,000} :$ $L_1^2\,(C_p\,W)\,\frac{1\,000}{C_p\ 1\,000}$ and $\delta_1 = \delta\,\frac{L^2}{L_1^2}\,\frac{C_p\ 1\,000}{C_p\ 1\,000}\,\frac{(W_1)}{(W)}$

Example.—What is the deflection caused by a weight of 12 000 lbs. upon a solid 1-in. square wrought-iron pillar 19.6 ins. long? $W =$ 26 000 lbs. and $C_p\ (W)\ 1\,000 = 0.00007_1$ $L^2 = 374.16$; $L_1^2 = 1\,505.44$; $C_p\ (W)\ 1\,000 = 0.00004$; $\delta = \frac{1}{6}$; hence $\delta_1 = \frac{1}{6}\,\frac{374.16}{1505.44}\,\frac{4}{7} = 0.02367$. Table No. 7 gives a deflection of 0.01828, hence the safe load given in Table No. 9, being 10 720 lbs., would be reduced by 388 lbs.

What is the deflection caused by 16 000 lbs. upon a solid cast-iron pillar 9.5 ins. long and 1 in. square as heretofore described? The breaking weight is 48 000 lbs. and the compression due to the last 1 000 lbs., as given in Table No. 1, column 3, is 0.00025. That of the last 1 000 lbs. of 16 000 lbs. is 0.000083, $L^2 = 90.25$ and $L_1^2 = 696.96$. The breaking deflection $\delta = \frac{1}{6}$ hence

$$\delta_1 = \frac{\delta\ L^2\ C_p\ W_1\ 1\ 000}{L_1^2\ C_p\ W\ 1\ 000} = \frac{1 \times\ 90.25 \times 0.000083}{6 \times 696.96 \times 0.000250} = 0.00835$$

which exceeds the deflection given in Table No. 7 (0.00800) by 0.00035, hence the safe load is reduced by 33.6 lbs.

An experimental test taken from the reports of the Watertown Arsenal confirms the correctness of deflections δ_1 of weights less than the breaking weight. The report of the Watertown Arsenal of 1888–89 gives on page 733 the deflections of a round hollow cast-iron pillar of 6.35 ins. outer and 4.24 ins. inner diameter under various loads when compressed in the testing machine. It ultimately failed by triple flexure. The length of the pillar is 13.09 or 25.3 diameters, hence the pillar considered as hinged at the ends which corresponds with it is 12.65 diameters long. The radius of gyration of this pillar expressed in terms of the diameter is found to be 0.030236 (in inches, 1.92), hence the length in terms of the radius of gyration is 40.9. Table No. 8 gives the safe load of 14 380 lbs. for this length. The deflection of a pillar 11.8 diameters long (which length in diameters, as given in Table No. 8, corresponds with a length in radius of gyration of 40.9) caused by a load of 14 380 lbs., may be computed by the formula $X : (C - W) = L^2 : L_1^2$ where L is the length of a pillar which has a breaking weight of 14 380 lbs. and which will be found by Table No. 1 to be 29.1; hence $X : 54\ 000^* = 139.24 : 846.8$ when $X = 8\ 877$ lbs. and $pX = 5\ 437$. Hence the radius of the pillar in question 11.8 diameters long bent by a load of 14 380 lbs. is 1 011 ins. and $\frac{L^2}{8\,R} = \delta_1 = \frac{139.24}{8\ 088} = 0.0172$.

The metal area of the pillar reported by the Watertown Arsenal is 17.28 sq. ins., hence the total load of the pillar corresponding to the safe load 14 380 lbs. is 248 486 lbs. The report gives deflections at the

* Breaking weight of a pillar 11.8 diameters long is $W = 38\ 000$ lbs.
$T = 16\ 000$ "
$W + T = 54\ 000$ lbs.

middle of the pillar in two directions, horizontal and vertical, for 240 000 lbs., of 0.04 and 0.06 in., respectively, and for 260 000 lbs., of 0.05 and 0.07 in. From this data may be deduced the deflection of 248 436 lbs. as 0.056 and 0.084 in. The true deflection the resultant of the vertical and horizontal deflections is $\delta_1 = \sqrt{0.056^2 + 0.084^2} = 0.101$. Divide this by the diameter of the pillar, 6.35, and it will be found that the deflection $\delta_1 = 0.016$.

Table No. 7 A gives the radii and deflections pertaining to various weights (W_1) from nil to 48 000 lbs., which is the breaking weight of a solid 1-in. square cast-iron pillar 9.5 ins. in length.

Table No. 7 B gives the radii and deflections pertaining to various weights (W) from nil to 26 000 lbs., which is the breaking weight of a solid 1-in. square wrought-iron pillar 19.6 ins. long.

If it is intended that the maximum strain of the pillar shall not exceed 16 000 lbs., one-sixth of the ultimate strength of the material, and that no load bending the pillar can be considered to be a safe load which results in a maximum strain greater than 16 000 lbs., it is clear that a load of 15 232 lbs. would certainly constitute a safe load, inasmuch as the resistance to the bending moment does not exceed 768 lbs. In fact, the deflection caused by a weight of 15 232 lbs. is less than 0.008, hence the bending moment is less than 128 lbs. It will be seen by Table No. 8 that the safe load for the pillar described previously is 15 300 lbs.

Table No. 8 has been computed in the manner indicated. A load W_1 is assumed as probable to constitute a safe load of a given pillar, and its deflection is computed by the method given on page 389; then the correctness of the assumed load is tested by the formula $W_1 + 6\delta_1 W_1 = MS$, wherein MS means the maximum strain at the intrados. If $MS > 16\,000$ lbs., then W_1 must be smaller than the W_1 first assumed, and if $MS < 16\,000$ lbs., then W_1 is greater. Column 1 in Table No. 8 gives the lengths of pillars expressed in diameters, and column 2 in terms of the radius of gyration, for convenient reference in computing safe loads for pillars of all forms. Column 3 gives the safe loads, and column 4 the respective breaking loads.

It will be noticed that for pillars less than seven diameters in length the safe load is one-fourth the breaking load, and less when the pillar is very short. From seven to about twelve diameters in length, the safe load varies from one-fourth to one-third of the breaking load. From

12 to 25 diameters the safe load varies from one-third to one-half the breaking load, and from 25 to 50 diameters the safe load is nearly one-half the breaking load. The significance of the relation of safe loads to breaking loads becomes more apparent by reversing the above statement as follows :

Breaking loads approximate safe loads (which cause a given maximum strain) in magnitude in a ratio related to the length of the pillar.

With a pillar of infinite length the safe load and the breaking weight must both be nil, and with a pillar of inappreciable length, say, a thin plate, the safe load multiplied by the factor of safety is equal to the breaking weight.

Table No. 9 gives for wrought-iron pillars all the data contained in Table No. 8 for cast-iron pillars.

Example.—Find from Table No. 8 the safe load per inch of metal of a round, hollow cast-iron column 20 ft. long and 12 ins. outer and 10 ins. inner diameter.

The radius of gyration is $R_g = \sqrt{\frac{D^2 + d^2}{16}} = \sqrt{\frac{144 + 100}{16}} = \frac{17}{4} =$ 4.25 ins. The length of the pillar in inches is $20 \times 12 = 240$ and length in terms of the radius of gyration is $L_g = 56.5$. Table No. 8 gives the safe load per inch of metal of a pillar 56.5 radii of gyration in length as 12 000 lbs.

It may be noted here that cast-iron pillars not exceeding 28 diameters or 97 radii of gyration in length are not subject to tension at the extrados under a safe load, as given in Table No. 8, when the load is placed in the center of gravity of the pillar. Inequalities and other probable defects of castings are assumed to be covered by the factor of safety.

Breaking loads for cast-iron pillars as given in Table No. 1 and for wrought-iron pillars as given in Table No. 2, also the respective safe loads, as given in Tables Nos. 8 and 9, being computed on the assumption that the load is applied in the center of gravity of the pillar, it is essential that this should be the case accurately, inasmuch as slight deviations cause material differences in their magnitude. A cast-iron pillar 10 ins. in diameter and 11.9 ft. in length ($L = 14.3$) will break under a load of 32 000 lbs. per inch metal area, when the load is placed in the center of gravity of the pillar. When placed 1 in. to one side of the center it will break under 21 150 lbs., and when placed 0.5 in. off the center of gravity of the pillar the breaking load is 26 050 lbs. or 19% less than when exactly in the center.

Brickwork.

The report of tests of the strength of structural material made at the Watertown Arsenal during the year 1884 gives on page 92 the compression of a common hard-brick pier, 8 ins. square and 16.48 ins. high, laid in lime mortar composed of 1 part lime and 3 parts sand, 15 months old when tested.

If brickwork laid up with good cement (say, 1 part of the best Rosendale and 2½ parts of sand) is used in ordinary building, at least six months expire before it is loaded to the full extent of the safe load assumed. The resistance to compression of brickwork so laid may be considered fully as great as that of brickwork laid up in lime mortar 15 months old. Its compressibility will probably be less. It is doubtless safe to assume that it is no greater.

Table No. 10 gives compression in parts of 1 for loads up to 2 000 lbs. per square inch, as deduced from the Watertown experiments, and shows a sufficient uniformity to warrant the acceptance of a modulus of elasticity of 160 000 without appreciable error in practice.

Let 2 000 lbs. be the breaking weight (it is 2 440 lbs. in the Watertown test) and 300 lbs. the maximum stress permissible under a safe load, also let the cohesion of the cement mortar be ignored as a medium of resistance to tension, then one-sixth the diameter becomes the greatest possible deflection at the breaking point. In other words, the breaking point occurs whenever the stress at the intrados amounts to 2 000 lbs. or the stress at the extrados is nil.

With uniform elasticity the radius $R = \frac{E}{2\,p\,(C - W)}$ and as $\delta = \frac{C - W}{6\,W}$ and $L^2 = 8\,\delta\,R$, hence, $L^2 = \frac{2\,E}{3\,p\,W}$ and as $E = 160\,000$, then $L^2 = \frac{174\,000}{W}$ and $W = \frac{174\,000}{L^2}$.

The stress at the intrados under a safe load is 300 lbs. up to a pillar length of 13.2 diameters, where the breaking load is 1 000 lbs. In pillars of greater lengths the stress at the intrados cannot exceed twice the amount of the safe load without causing tension at the extrados.

For lengths in terms of the diameters, L, the square of the respective lengths, L^2, breaking loads per square inch of sectional area, breaking deflections, safe loads per inch area and deflections under them, see Table No. 11.

It will be noticed that for lengths greater than 13.2 diameters the deflection of safe loads is intended to remain constant.

The weights of floors supported in the center of resistance of a wall or pier constitute a part of the superincumbent weight, pound for pound. It frequently happens, however, that floor beams or girders are so placed that they constitute an eccentric load, and if n be the eccentricity, such floor loads should be multiplied by $(6\,n + 1)$ in order to ascertain the value of the centric load which represents them. This rule is approximately correct in view of the minute deflections under safe loads as applied in practical building.

Yellow Pine Pillars.

Table No. 12 gives the safe loads for yellow pine pillars based upon $E = 1\,600\,000$, computed as described previously.

For knots and sap due allowance is to be made, and 25% may be added for pillars for temporary use.

Summary.

For convenience in use, the leading formulas deduced in the preceding discussion are printed here, together with the notation.

Notation.—C is the ultimate resistance of the material to compression.

T is the ultimate resistance of the material to tension.

W " breaking weight.

W_1 is a weight less than the breaking weight.

δ is the breaking deflection.

δ_1 " deflection caused by W_1.

R " radius at the breaking point.

R_1 " " with the load W_1.

L " length of the pillar in terms of the diameter.

d " diameter, taken as 1 in the paper. Hence the moment of inertia I_1 is $\frac{1}{12}$ for a solid square section, and the moment of resistance is $\frac{1}{6}$.

C_p is the amount of compression.

R_g " radius of gyration.

p " coefficient 0.6125.

E_c is the modulus of elasticity for compression.
E_t " " " tension.
n " eccentricity, expressed by a fraction of the diameter.
x and y are the coefficients of stress in eccentric loads.

Formulas.—Breaking weights and safe loads are computed upon the assumption that pillars are compressed in the center of resistance and are subject to single flexure, except in the case of eccentric loading.

1. Deflection at the breaking point.

$$\delta = \frac{C - W}{6\ W}$$

2. Radius at the breaking point corresponding to given deflections.

$$R = \frac{2 - (C_p\ (W + p\ (C - W)) + C_p\ (W - p\ (C - W)))}{2\ (C_p\ (W + p\ (C - W)) - C_p\ (W - p\ (C - W)))}$$

3. Breaking length.

$$L = \sqrt{8\ \delta\ R}$$

With eccentric loading.

4. Compression at the ends of the pillar.

$$x = 6\,n + 1 \qquad y = 6\,n - 1.$$

5. Breaking weight expressed in ultimate resistance of the material and centric breaking loads.

$$W_1 = \pm\sqrt{\left(\frac{C + W + 6\,n\ W}{1}\right)^2 - C\ W} + \left(\frac{C + W + 6\,n\ W}{2}\right)$$

$$W_1 = \pm\sqrt{\left(\frac{W - 6\,n\ W - T}{2}\right)^2 + T\ W} + \left(\frac{W - 6\,n\ W - T}{2}\right)$$

6. Resistance, to bending when W_1 is less than W.

$$X = (C - W)\frac{L^2}{L_1^2}$$

7. Deflection, when W_1 is less than W.

$$\delta_1 = \frac{(C - W)\ L^4}{6\ E\ L_1^2} \text{ or } \delta_1 = \delta\ \frac{L^2\ C_p\ 1\,000\ (W_1)}{L_1^2\ C_p\ 1\,000\ (W)}$$

8. Brickwork.

$$E = 160\,000. \quad W = \frac{2\ E}{3\ p\ L^2} = \frac{174\,000}{L^2} \quad R = \frac{E}{2\ p\ (C - W)} = \frac{130\ 60\ 0}{C - W}.$$

9. Yellow pine posts.

$$E = 1\,600\,000 \qquad W_1 = \frac{1\,740\,000}{L^2} \qquad R = \frac{1\,306\,000}{C - W}.$$

TABLE No. 1.—Breaking Weights for Round-Ended Solid 1-In. Square Cast-Iron Pillars, Loaded in the Center.

Breaking weight, W.	Compression due to W.	Compression due to the last 1 000 pounds.	Strain at the middle of the column. Compression, —; Tension, +	Deflection, δ.	Average strain at the intrados, $W + p(C - W)$.	Average strain at the extrados, $W - p(C - W)$.	Radius, R.	Length, L.
96 000	0.026000	.....	—96 000	.....	96 000	96 000	.....	.....
95 000	0.025333	0.000667	—94 000	0.00175	95 612	94 388	1 259.2	4.2
94 000	0.024736	0.000597	—92 000	0.00345	95 223	92 775	670.2	4.2
93 000	0.024155	0.000581	—90 000	0.00537	94 837	91 163	462.9	4.4
92 000	0.023590	0.000565	—88 000	0.00724	94 449	89 551	354.6	4.4
91 000	0.023038	0.000552	—86 000	0.00916	94 062	87 938	288.8	4.5
90 000	0.022493	0.000545	—84 000	0.01111	93 675	85 325	244.2	4.6
89 000	0.021955	0.000538	—82 000	0.01311	93 287	84 713	213.2	4.7
88 000	0.021424	0.000531	—80 000	0.01515	92 900	83 100	189.1	4.8
87 000	0.020900	0.000524	—78 000	0.01724	92 512	81 488	170.5	4.8
86 000	0.020383	0.000517	—76 000	0.01938	92 125	79 875	155.7	4.9
85 000	0.019873	0.000510	—74 000	0.02157	91 737	78 263	143.7	4.9
84 000	0.019370	0.000503	—72 000	0.02381	91 350	76 650	133.5	5.0
83 000	0.018874	0.000496	—70 000	0.02610	90 962	75 038	125.1	5.1
82 000	0.018385	0.000489	—68 000	0.02846	90 575	73 425	117.8	5.1
81 000	0.017903	0.000482	—66 000	0.03086	90 187	71 813	111.6	5.2
80 000	0.017428	0.000475	—64 000	0.03333	89 800	70 200	106.3	5.3
79 000	0.016960	0.000468	—62 000	0.03797	89 412	68 588	101.5	5.5
78 000	0.016499	0.000461	—60 000	0.04060	89 025	66 975	97.9	5.6
77 000	0.016045	0.000454	—58 000	0.04113	88 637	65 363	93.8	5.6
76 000	0.015598	0.000447	—56 000	0.04386	88 250	63 756	90.6	5.6
75 000	0.015158	0.000440	—54 000	0.04666	87 862	62 138	87.7	5.7
74 000	0.014725	0.000433	—52 000	0.05000	87 475	60 525	85.1	5.8
73 000	0.014299	0.000426	—50 000	0.05251	87 087	58 913	82.3	5.9
72 000	0.013880	0.000419	—48 000	0.05585	86 700	57 300	80.7	6.0
71 000	0.013468	0.000412	—46 000	0.05879	86 312	55 688	78.8	6.2
70 000	0.013063	0.000405	—44 000	0.06190	85 855	54 145	77.5	6.2
69 000	0.012665	0.000398	—42 000	0.06522	85 537	52 463	75.6	6.2
68 000	0.012274	0.000391	—40 000	0.06872	85 150	50 850	74.3	6.2
67 000	0.011890	0.000384	—38 000	0.07211	84 762	49 238	73.1	6.4
66 000	0.011513	0.000377	—36 000	0.07575	84 375	47 625	72.0	6.6
65 000	0.011143	0.000370	—34 000	0.07949	83 987	46 013	71.0	6.7
64 000	0.010718	0.000362	—32 000	0.08333	83 600	44 400	70.2	6.8
63 000	0.010426	0.000355	—30 000	0.08730	83 212	42 788	69.5	6.9
62 000	0.010078	0.000348	—28 000	0.09140	82 825	41 175	68.9	7.0
61 000	0.009737	0.000341	—26 000	0.09590	82 437	39 563	68.4	7.2
60 000	0.009403	0.000334	—24 000	0.10000	82 050	37 950	68.1	7.3
59 000	0.009076	0.000327	—22 000	0.10482	81 662	36 338	67.6	7.5
58 000	0.008756	0.000320	—20 000	0.10920	81 275	34 725	67.5	7.6
57 000	0.008443	0.000313	—18 000	0.11404	80 887	33 113	67.3	7.8
56 000	0.008137	0.000306	—16 000	0.11905	80 500	31 500	67.2	8.0
55 000	0.007838	0.000299	—14 000	0.12406	80 112	29 888	67.2	8.1
54 000	0.007546	0.000293	—12 000	0.13000	79 725	28 275	67.2	8.3
53 000	0.007261	0.000285	—10 000	0.13522	79 337	26 663	67.3	8.5
52 000	0.006983	0.000278	— 8 000	0.14102	78 950	25 050	67.4	8.7
51 000	0.006712	0.000271	— 6 000	0.14706	78 562	23 438	67.4	8.9
50 000	0.006448	0.000264	— 4 000	0.15333	78 175	21 825	67.5	9.1
49 000	0.006191	0.000257	— 2 000	0.15986	77 787	20 213	67.7	9.3
48 000	0.005941	0.000250	0 000	0.16666	77 400	18 600	67.7	9.5
47 000	0.005698	0.000243	+ 2 000	0.17375	77 012	16 988	67.8	9.7
46 000	0.005460	0.000238	+ 4 000	0.18116	76 625	15 375	68.0	9.9
45 000	0.005232	0.000228	+ 6 000	0.18888	76 237	13 763	68.1	10.1
44 000	0.005014	0.000218	+ 8 000	0.19700	75 850	12 150	68.3	10.2
43 000	0.004806	0.000208	+10 000	0.20542	75 462	10 538	68.5	10.6
42 000	0.004608	0.000198	+12 000	0.21429	75 075	8 925	68.6	10.8
41 000	0.004418	0.000190	+14 000	0.22358	74 687	7 313	68.7	11.1
40 000	0.004235	0.000183	+16 000	0.23333	74 300	5 700	68.9	11.3

TABLE No. 1—(*Continued*).

Breaking weight, W.	Compression due to W.	Compression due to the last 1 000 pounds.	Strain at the middle of the column. Compression, —; Tension, +	Deflection, δ.	Average strain at the intrados. $W + p\,(C - W)$.	Average strain at the extrados. $W - p\,(C - W)$.	Radius, R.	Length, L.
39 000	0.004059	0.000176	*—94 000	0.23500	72 687	5 313	72.2	11.6
38 000	0.003890	0.000169	—92 000	0.23684	71 075	4 925	74.3	11.8
37 000	0.003728	0.000162	—90 000	0.23874	69 462	4 538	79.8	12.3
36 000	0.003533	0.000155	—88 000	0.24074	67 850	4 150	83.7	12.7
35 000	0.003425	0.000148	—86 000	0.24285	66 237	3 763	87.9	13.0
34 000	0.003283	0.000142	—84 000	0.24510	64 625	3 375	93.2	13.5
33 000	0.003147	0.000136	—82 000	0.24747	63 012	2 988	97.6	13.9
32 000	0.003016	0.000131	—80 000	0.25000	61 400	2 600	103.3	14.3
31 000	0.002891	0.000125	—78 000	0.25268	59 787	2 213	109.1	14.8
30 000	0.002771	0.000120	—76 000	0.25555	58 175	1 825	115.3	15.3
29 000	0.002657	0.000114	—74 000	0.25862	56 562	1 438	121.5	15.8
28 000	0.002547	0.000110	—72 000	0.26190	54 950	1 050	128.7	16.4
27 000	0.002441	0.000106	—70 000	0.26543	53 337	663	136.4	17.0
26 000	0.002337	0.000104	—68 000	0.26920	51 725	275	144.7	17.6
25 000	0.002235	0.000102	—66 000	0.27333	50 112	112	153.7	18.3
24 000	0.002134	0.000101	—64 000	0.27777	48 500	500	170.4	19.4
23 000	0.002034	0.000100	—62 000	0.28261	46 887	887	173.8	19.8
22 000	0.001935	0.000099	—60 000	0.28800	45 175	1 175	185.8	20.6
21 000	0.001837	0.000098	—53 000	0.29365	43 662	1 662	196.5	21.5
20 000	0.001740	0.000097	—56 000	0.30000	42 050	2 050	208.9	22.4
19 000	0.001644	0.000096	—54 000	0.30700	40 437	2 437	221.4	23.3
18 000	0.001549	0.000095	—52 000	0.31480	38 825	2 825	234.6	24.3
17 000	0.001455	0.000094	—50 000	0.32335	37 212	3 212	248.5	25.3
16 000	0.001362	0.000093	—48 000	0.33333	35 600	3 600	262.3	26.4
15 000	0.001270	0.000092	—46 000	0.34444	33 987	3 987	276.9	27.6
14 000	0.001179	0.000091	—44 000	0.35715	32 375	4 375	291.7	28.8
13 000	0.001089	0.000090	—42 000	0.37154	30 772	4 772	306.6	30.1
12 000	0.001000	0.000089	—40 000	0.38888	29 150	5 150	322.2	31.8
11 000	0.000912	0.000088	—38 000	0.40909	27 537	5 537	337.8	33.2
10 000	0.000825	0.000087	—36 000	0.43333	25 925	5 925	353.9	35 0
9 000	0.000739	0.000086	—34 000	0.46300	24 312	6 312	371.3	37.0
8 000	0.000654	0.000085	—32 000	0.50000	22 700	6 700	389.7	39.4
7 000	0.000570	0.000084	—30 000	0.54760	21 087	7 087	409.9	42.3
6 000	0.000487	0.000083	—28 000	0.61111	19 775	7 475	425.9	45.6
5 000	0.000404	0.000083	—26 000	0.70000	17 832	7 832	460.8	50.0
4 000	0.000322	0.000082	—24 000	0.83333	16 250	8 250	486.0	56.9
3 000	0.000240	0.000082	—22 000	1.05555	14 637	8 637	521.0	66.3
2 000	0.000195	0.000081	—20 000	1.50000	13 025	9 025	535.7	80.0
1 000	0.000079	0.000080	—18 000	2.83330	11 412	9 412	584.4	115.1

* Strains at this point change from the extrados to the intrados, where heretofore the strain has been 96 000. At the extrados the strain continues at +16 000 to the end of the table.

TABLE No. 2.—Breaking Weights for Round-Ended Solid 1-In. Square Wrought-Iron Pillars, Loaded in the Center.

Breaking weight, *W*.	Compression due to *W*.	Compression due to the last 1 000 pounds.	Strain at the middle of the column. Compression, —; Tension, +	Deflection, δ.	Average strain at the extrados.	Average strain at the intrados.	Radius, *R*.	Length, *L*.
52 000	0.00800	.....	—52 000	.....	—52 000	52 000	.....	.....
51 000	0.00749	0.00051	—50 000	0.0033	—50 388	51 612	1 654.2	6.6
50 000	0.00701	0.00048	—48 000	0.0066	—48 776	51 224	863.4	6.7
49 000	0.00655	0.00046	—46 000	0.0102	—47 163	50 837	605.7	7.0
48 000	0.00611	0.00044	—44 000	0.0140	—45 550	50 450	473.2	7.4
47 000	0.00569	0.00042	—42 000	0.0177	—43 938	50 062	394.0	7.4
46 000	0.00529	0.00040	—40 000	0.0217	—42 325	49 675	346.5	7.7
45 000	0.00491	0.00038	—38 000	0.0259	—40 713	49 287	313.8	8.0
44 000	0.00455	0.00036	—36 000	0.0303	—39 100	48 900	290.1	8.4
43 000	0.00421	0.00034	—34 000	0.0349	—37 488	48 512	273.3	8.7
42 000	0.00389	0.00032	—32 000	0.0397	—35 875	48 125	261.4	9.1
41 000	0.00359	0.00030	—30 000	9.0447	—34 263	47 737	254.7	9.5
40 000	0.00331	0.00028	—28 000	0.0500	—32 650	47 350	250.0	10.0
39 000	0.00305	0.00026	—26 000	0.0555	—31 038	46 962	250.0	10.5
38 000	0.00281	0.00024	—24 000	0.0614	—29 425	46 575	250.0	11.1
37 000	0.00259	0.00022	—22 000	0.0676	—27 813	46 187	250.0	11.6
36 000	0.00239	0.00020	—20 000	0.0741	—26 200	45 800	250.0	12.1
35 000	0.00221	0.00018	—18 000	0.0810	—24 588	45 412	253.6	12.8
34 000	0.00205	0.00016	—16 000	8.0884	—22 975	45 025	257.6	13.5
33 000	0.00191	0.00014	—14 000	0.0960	—21 363	44 637	261.7	14.1
32 000	0.00178	0.00013	—12 000	0.1042	—19 750	44 250	265.0	14.8
31 000	0.00166	0.00012	—10 000	0.1129	—18 128	43 872	270.4	15.6
30 000	0.00155	0.00011	— 8 000	0.1222	—16 525	43 475	272.1	16.3
29 000	0.00145	0.00010	— 6 000	0.1322	—14 913	43 087	277.8	17.1
28 000	0.00136	0.00009	— 4 000	0.1426	—13 400	42 600	284.2	18.0
27 000	0.00128	0.00008	— 2 000	0.1543	—11 688	42 312	286.0	18.7
26 000	0.00121	0.00007	0 000	0.1666	—10 075	41 925	288.4	19.6
25 000	0.00115	0.00006	+ 2 000	0.1800	— 8 463	41 537	292.6	20.5
24 000	0.00110	0.00005	+ 4 000	0.1944	— 6 850	41 150	295.4	21.4
23 000	0.00105	0.00005	+ 6 000	0.2101	— 5 238	40 762	301.5	22.5
22 000	0.00100	0.00005	+ 8 000	0.2272	— 3 625	40 375	306.2	23.5
21 000	0.00095	0.00005	+10 000	0.2460	— 2 013	39 987	309.0	24.6
20 000	0.00090	0.00005	+12 000	0.2666	— 400	39 600	310.0	25.7
19 000	0.00085	0.00005	+14 000	0.2900	+ 1 212	39 212	318.0	27.1
18 000	0.00080	0.00005	+16 000	0.3148	+ 2 825	38 825	320.0	28.4
17 000	0.00075	0.00005	+18 000	0.3431	+ 4 437	38 437	323.5	29.7
16 000	0.00070	0.00005	+20 000	0.3750	+ 6 050	38 050	326.3	31.3
15 000	0.00065	0.00005	+22 000	0.4111	+ 7 662	37 662	328.0	32.8
14 000	0.00060	0.00005	+24 000	0.4524	+ 9 275	37 275	329.6	34.5
13 000	0.00055	0.00005	+26 000	0.5000	+10 887	36 887	331.8	36.4
12 000	0.00050	0.00004	+28 000	0.5555	+12 500	36 500	332.0	38.4
11 000	0.00046	0.00004	+30 000	0.6212	+14 112	36 112	329.9	40.5
10 000	0.00041	0.00004	+32 000	0.7000	+15 725	35 725	329.7	42.9
9 000	0.00037	0.00004	+34 000	0.8000	+17 337	35 337	327.6	45.7
8 000	0.00032	0.00004	+36 000	0.9166	+18 950	34 950	326.6	48.9
7 000	0.00028	0.00004	+38 000	1.0714	+20 562	34 562	325.5	52.8
6 000	0.00024	0.00004	+40 000	1.2777	+22 175	34 175	323.5	57.5
5 000	0.00020	0.00004	+42 000	1 5666	+23 778	33 778	322.4	63.5
4 000	0.00016	0.00004	+44 000	2.0000	+25 400	33 400	318.3	71.3
3 000	0.00012	0.00004	+46 000	2.7222	+27 010	33 010	313.4	82.5
2 000	0.00008	0.00004	+48 000	4.1666	+28 625	32 625	304.8	100.7
1 000	0.00004	0.00004	+50 000	8.5000	+30 240	32 240	295.0	141.6
500	0.00002	0.00004	+51 000	17.1666	+32 040	33 040	280.9	196.4

TABLE No. 3.—Breaking Loads of Round-Ended Solid 1-In. Square Cast-Iron Pillars, Loaded in the Center.

Compression, — Tension, +

Breaking weight, W.	Strains at the middle of the pillar at extrados to *, afterward at intrados.	Deflection, δ.	Average strain at the intrados, $\frac{W+p(C-W)}{}$.	Average strain at the extrados, $\frac{W-p(C-W)}{}$.	Radius, R.	Length, L.
96 000	—96 000	.....	.....	.....	.....	.....
95 000	—94 000	0.0020	95 635	94 365	1 230.8	4.0
94 000	—92 000	0.0040	95 273	92 727	643.7	4.4
93 000	—90 000	0.0064	94 909	91 091	455.5	4.8
92 000	—88 000	0.0088	94 545	89 455	349.1	4.9
91 000	—86 000	0.0104	94 182	87 818	268.6	4.8
90 000	—84 000	0.0124	93 675	86 275	243.1	4.8
89 000	—81 630	0.0144	93 287	84 486	.....	.....
88 000	—79 250	0.0168	92 900	82 640	.....	.....
87 000	—76 730	0.0184	92 512	80 710	.....	.....
86 000	—74 300	0.0200	92 125	78 834	.....	.....
85 000	—72 150	0.0220	91 737	77 130	133.5	4.8
84 000	—70 000	0.0240	91 350	75 425	.....	.....
83 000	—67 500	0.0265	90 962	73 510	.....	.....
82 000	—65 000	0.0290	90 575	71 590	.....	.....
81 000	—62 500	0.0305	90 187	69 670	.....	.....
80 000	—60 000	0.0324	89 800	67 750	96.4	5.0
79 000	—57 500	0.0344	89 412	65 220	.....	.....
78 000	—55 000	0.0368	89 075	63 910	.....	.....
77 000	—52 500	0.0400	88 640	62 000	.....	.....
76 000	—50 000	0.0432	88 250	60 075	81.2	5.3
75 000	—46 800	0.0480	87 826	57 730	77.8	5.4
74 000	—43 500	0.0528	87 475	54 700	73.5	5.5
73 000	—39 500	0.0580	87 090	52 480	71.3	5.7
72 000	—35 000	0.0632	86 700	49 338	.....	.....
71 000	—30 400	0.0697	86 312	46 133	65.9	6.0
70 000	—25 300	0.0762	85 925	42 620	.....	.....
69 000	—19 700	0.0826	85 537	38 800	61.2	6.3
68 000	—14 000	0.0900	85 150	34 925	59.8	6.5
67 000	—10 000	0.0975	84 762	32 090	.....	.....
66 000	— 6 000	0.1000	84 375	29 250	.....	.....
65 000	— 2 200	0.1054	83 987	26 535	58.6	7.0
64 000	+ 1 000	0.1104	83 600	24 190	58.1	7.1
63 000	+ 4 500	0.1144	83 170	21 660	58.0	7.2
62 000	+ 8 000	0.1184	82 825	19 125	.....	.....
61 000	+11 500	0.1224	82 437	16 590	57.7	7.5
60 000	+15 000	0.1264	82 050	14 063	57.5	7.6
59 000	+16 000	0.1304	81 540	13 060	.....	.....
	*				.....	.....
58 000	—94 600	0.1344	80 420	12 675	.....	.....
57 000	—93 290	0.1372	79 230	12 290	58.5	8.0
56 000	—91 975	0.1400	78 035	11 900	63.8	8.4
55 000	—90 670	0.1432	76 850	11 510	66.0	8.7
54 000	—89 350	0.1464	75 650	11 125	68.3	8.9
53 000	—88 040	0.1492	74 460	10 740	70.6	9.1
52 000	—86 725	0.1520	73 270	10 350	71.6	9.3
51 000	—85 420	0.1540	72 080	9 960	75.7	9.6
50 000	—84 100	0·1560	70 890	9 575	79.0	9.9
49 000	—82 640	0.1572	69 500	9 190	82.1	.....
48 000	—81 175	0.1581	68 320	8 800	85.1	10.0
47 000	—80 020	0.1608	67 225	8 410	88.0	10.6
46 000	—78 850	0.1632	66 120	8 025	91.1	10.9
45 000	—77 540	0.1652	64 930	7 640	95.3	11.2
44 000	—76 225	0.1672	63 740	7 250	99.3	11.5
43 000	—74 920	0.1676	62 550	6 860	102.4	11.7

TABLE No. 3—(*Continued*).

Breaking weight, W.	Strains at the middle of the pillar at extrados to*, afterward at intrados.	Deflection, δ.	Average strain at the intrados, $W + p(C - W)$.	Average strain at the extrados, $W - p(C - W)$.	Radius, R.	Length, L.
42 000	—73 600	0.1680	61 350	6 475	106.5	11.9
41 000	—72 290	0.1692	60 165	6 090	112.1	12.3
40 000	—70 975	0.1704	58 970	5 700	115.7	12.5
39 000	—69 700	0.1720	57 800	5 310	120.5	12.8
38 000	—68 350	0.1736	56 590	4 925	125.7	13.2
37 000	—66 040	0.1760	54 790	4 540	134.4	13.8
36 000	—65 725	0.1784	54 200	4 150	137.0	14.0
35 000	—64 420	0.1808	53 020	3 760	143.0	14.4
34 000	—63 100	0.1824	51 820	3 375	149.5	14.7
33 000	—61 790	0.1856	50 630	2 990	156.1	15.2
32 000	—60 475	0.1888	49 440	2 600	164.4	15.5
31 000	—59 170	0.1928	48 250	2 210	172.1	16.4
30 000	—57 850	0.1968	47 060	1 825	179.1	16.8
29 000	—56 540	0.2004	45 970	1 440	187.0	17.0
28 000	—55 230	0.2044	44 730	1 050	196.2	17.8
27 000	—53 920	0.2084	43 490	660	205.4	18.5
26 000	—52 600	0·2128	42 290	375	215.1	19.1
25 000	—51 300	0.2192	41 110	112	224.4	19.8
24 000	—50 000	0.2256	39 920	500	234.2	20.5
23 000	—48 500	0.2336	38 620	890	245.2	21.4
22 000	—47 000	0.2416	37 090	1 275	260.0	22.4
21 000	—46 000	0.2504	36 310	1 660	269.0	23.2
20 000	—45 000	0.2592	35 310	2 050	275.0	23.9
19 000	—44 250	0.2708	34 460	2 470	282.3	24.7
18 000	—43 500	0.2824	33 620	2 825	290.5	25.6
17 000	—42 550	0.2980	32 650	3 210	298.2	26.6
16 000	—41 600	0.3136	31 680	3 600	306.9	27.7
15 000	—40 800	0.3316	30 800	3 990	312.9	28.8
14 000	—40 000	0.3496	29 910	4 375	320.4	29.9
13 000	—39 000	0.3724	28 910	4 762	329.4	31.3
12 000	—38 000	0.3953	27 910	5 150	338.4	32.7
11 000	—36 650	0.4320	26 710	5 540	350.2	34.8
10 000	—35 300	0.4688	25 500	5 925	362.5	36.9
9 000	—33 650	0.5128	24 100	6 310	374.4	39.9
8 000	—32 000	0.5568	22 700	6 700	390.6	41.7
7 000	—30 150	0.6160	21 180	7 090	410.1	44.9
6 000	—28 300	0.6752	19 660	7 475	431.3	48.2
5 000	—26 650	0.7504	18 260	7 860	448.7	51.9
4 000	—25 000	0.8256	16 860	8 239	473.5	55.9

TABLE No. 4.—Breaking Weights of Round-Ended Solid 1-In. Square Wrought-Iron Pillars, Loaded in the Center.

Compression, — Tension, +

Breaking weight, W.	Strains at the middle of the pillar.	Deflection, δ.	Average strain at the intrados, $W + p(C - W)$.	Average strain at the extrados, $W - p(C - W)$.	Radius, R.	Length, L.
52 000	52 000					
51 000	49 900	0.0040	51 610	50 330	1 600.8	7.1
50 000	47 800	0.0080	51 225	48 650	827.5	7.2
49 000	45 500	0.0120	50 840	46 860	558.1	7.3
48 000	43 000	0.0160	50 450	44 940	424.7	7.3
47 000	40 600	0.0208	50 030	42 960	351.4	7.6
46 000	38 200	0.0256	49 675	41 220	310.3	7.7
45 000	36 100	0.0304	49 290	39 550	285.2	8.3
44 000	34 000	0.0352	48 900	37 875	268.3	8.6
43 000	31 500	0.0428	48 512	35 960	252.0	9.2
42 000	28 000	0.0504	48 125	33 425	337.7	9.7
41 000	25 000	0.0600	47 740	31 200	230.6	10.5
40 000	20 500	0.0696	47 350	28 160	222.9	11.1
39 000	15 000	0.0788	46 960	24 300	218.8	11.7
38 000	4 000	0.0890	46 575	12 275	199.0	11.8
37 000	12 000	0.0996	46 190	6 999	195.9	12.2
36 000	16 000	0.1112	45 800	4 150	197.7	13.2
35 000	19 600	0.1228	45 410	1 430	199.5	14.0
34 000	22 000	0.1344	45 025	300	202.4	14.7
33 000	24 000	0.1488	44 640	2 910	204.4	14.7
32 000	25 500	0.1632	44 250	3 220	210.0	16.5
31 000	27 000	0.1812	43 860	4 525	213.8	17.6
30 000	28 000	0.1992	43 475	5 525	218.8	18.6
29 000	29 400	0.2164	43 090	6 770	226.3	19.7
28 000	30 000	0.2336	42 700	7 525	227.8	20.6
27 000	31 000	0.2522	42 310	8 525	232.1	21.6
26 000	31 500	0.2708	41 925	9 220	237.7	22.7
25 000	31 750	0.2952	41 540	9 760	248.6	24.0
24 000	32 000	0.3169	41 150	10 300	249.3	25.1
23 000	32 250	0.3440	40 760	10 840	255.3	26.5
22 000	32 500	0.3684	40 375	11 380	261 3	27.7
21 000	33 000	0.3932	39 990	12 075	267.0	28.9
20 000	33 500	0.4180	39 600	12 770	271.3	30.1
19 000	33 750	0.4462	39 210	13 310	279 7	31.6
18 000	34 000	0.4744	38 825	13 850	285.3	32.9
17 000	34 500	0.5299	38 440	14 240	292.2	35.1
16 000	35 000	0.5855	38 050	15 240	297.3	37.3
15 000	35 500	0.6419	37 660	15 930	302.7	39.4
14 000	36 000	0.6983	37 275	16 625	308.3	41.5
13 000	36 500	0.7503	36 890	17 320	311.0	43.2
12 000	37 000	0.8023	36 500	18 010	319.2	45.2
11 000	37 500	0.8934	36 110	18 700	325.4	48.2
10 000	38 000	0.9846	35 725	19 400	330.0	50.9
9 000	38 300	1.0374	35 340	19 970	334.2	52.6
8 000	38 600	1.0902	34 950	20 540	340.0	54.4
7 000	39 300	1.3979	34 560	21 360	343.4	62.0
6 000	40 000	1.7056	34 175	22 175	346.2	68 7
5 000	40 750	2.2910	33 790	23 020	348.9	80.0
4 000	40 500	2.7526	33 400	23 870	351.8	88.0

TABLE No. 5.—Eccentric Breaking Loads for Various Eccentricities (n = a Fraction of the Diameter) of Round-Ended Solid 1-In. Square Cast-Iron Pillars.

$$W_1 = \pm\sqrt{\left(\frac{W - T - 6\,n\,W}{2}\right)^2 + T\,W} + \frac{W - T - 6\,n\,W}{2}$$

Central breaking loads, W.	Length, L.	Breaking loads for—				
		$n = 0.1$.	$n = 0.3$.	$n = 0.5$.	$n = 0.7$.	$n = 1.0$.
84 000	5.0	46 500	13 840	7 040	4 640	3 060
72 000	6.0	45 130	13 260	7 000	4 590	3 040
62 000	7.0	41 500	12 670	6 760	4 530	3 010
56 000	8.0	36 680	12 260	6 650	4 480	3 000
50 000	9.1	30 350	11 800	6 530	4 430	2 980
45 000	10.1	27 850	11 360	6 400	4 380	2 950
41 000	11.1	25 930	10 970	6 290	4 330	2 930
37 000	12.3	23 800	10 540	6 150	4 270	2 900
35 000	13.0	22 680	10 310	6 080	4 250	2 880
32 000	14.3	21 150	10 000	6 000	4 200	2 880
30 000	15.3	20 000	9 670	5 860	4 130	2 840
27 000	17.0	18 350	9 220	5 700	4 050	2 800
24 000	19.4	16 600	8 770	5 470	3 960	2 760
21 000	21.5	14 920	8 190	5 300	3 940	2 710
17 000	25.3	12 500	7 360	4 950	3 660	2 620
13 000	30.1	10 000	6 350	4 470	3 410	2 500
10 000	35.0	8 000	5 430	4 000	3 120	2 340
5 000	50.0	4 350	3 000	2 700	2 160	1 860

TABLE No. 6.—Eccentric Breaking Loads for Various Eccentricities (n = a Fraction of the Diameter) of Round-Ended Solid 1-In. Square Wrought-Iron Pillars.

$$W_1 = \pm\sqrt{\left(\frac{C + W + 6\,n\,W}{2}\right)^2 - C\,W} + \frac{C + W + 6\,n\,W}{2}$$

Central breaking load, W.	Length, L.	Breaking loads for—				
		$n = 0.1$.	$n = 0.3$.	$n = 0.5$.	$n = 0.7$.	$n = 1.0$.
49 000	7.0	23 930	17 960	11 850	8 550	6 560
40 000	10.0	22 170	13 860	10 310	8 270	6 320
36 000	12.1	21 180	13 400	10 000	8 100	6 290
30 000	16.3	19 340	12 650	9 672	7 790	6 100
25 000	20.5	17 430	11 800	9 100	7 450	5 980
21 000	24.6	15 600	10 930	8 570	7 090	5 640
18 000	28.4	14 020	10 150	8 070	6 740	5 430
16 000	31.3	12 470	9 480	7 120	6 460	5 240
14 000	34.8	12 090	8 840	6 930	6 100	5 030
12 000	38.3	10 240	8 050	6 690	5 740	4 710
10 000	43.9	8 780	7 140	6 000	5 270	4 420

TABLE No. 7 A.—A Round-Ended Solid 1-In. Square Cast-Iron Pillar. $W = 48\ 000$ Lbs., $R = 67.7$ Ins., $\delta = \frac{1}{6}$. When Loaded in the Center with Various Weights $W_1 < W$.

Bending weights, W_1.	$\frac{L^2}{L_1^2}$	X.	pX.	R_1	δ_1	Compression due to bending moment.	$\frac{X}{6}$
48 000	1.00000	48 000	29 400	67.7	0.16666	0.16666	8 000
44 000	.86745	41 637	25 500	88.2	0.12812	0.10000	5 637
40 000	.70679	33 926	20 780	124.9	0.09047	0.05384	3 620
39 070	.6742	32 360	19 820	135.4	0.08333	0.04065	3 256
36 000	.55961	26 860	16 352	185.2	0.06100	0.02700	2 200
32 000	.44134	21 800	12 975	278.4	0.04059	0.01300	1 300
28 000	.33555	16 106	9 855	442.8	0.02552	0.00600	714
24 000	.23979	11 510	7 050	690.0	0.01655	0.00460	397
20 000	.17986	8 613	5 296	977.7	0.01155	0.00177	231
16 000	.12949	6 215	3 807	1 418.5	0.00800	0.00090	128
12 000	.08914	4 280	2 620	2 157.6	0.00528	0.00046	63
8 000	.05913	2 790	1 710	3 458.0	0.00330	0.00015	26
4 000	.02787	1 338	820	7 300.0	0.00155	0.00004	6.2
1 000	.00681	327	200	31 232.0	0.00036	0.00000	0.36

At the breaking point the deflection is 0.16666; hence the bending moment is 48 000 × 0.1666 = 8 000. To bend the pillar to half the breaking deflection, the bending weight is 39 070 lbs., and the bending moment 3 256 lbs. An endwise compression of 24 000 lbs. (one-half the breaking weight) results in a deflection of 0.01655, about $\frac{1}{10}$ of the breaking deflection and the bending moment is 24 000 × 0.01655 = 397, or about the twentieth part of the bending moment at the breaking point.

When the load is 16 000 lbs. the deflection is reduced to 0.008, and the bending moment is 16 000 × 0.008 = 128 lbs., about the sixtieth part of the bending moment at the point of breaking. It may be observed here that the resistance to bending in this case being six times the bending moment, 6 × 128 = 768 lbs., hence the strain at the intrados is 16 768 lbs., consisting of the strain caused by resistance to the bending moment and that of vertical compression caused by the bending weight.

TABLE No. 7 B.—A Solid Round-Ended 1-In. Square Wrought-Iron Pillar, 19.6 Ins. Long, Loaded in the Center with Various Weights $W_1 < W$.

W_1	L.	L^2	$\frac{L_2}{L_1^2}$	pX.	X.	δ_1
26 000	19.6	374.16	.....	15 926	26 000	0.16666
24 000	21.4	457.96	0.81701	13 010	21 240	0.10300
22 000	23.5	552.25	0.67752	10 790	17 600	0.07224
20 000	25.8	665.64	0.56210	8 950	14 610	0.04758
18 000	28.4	806.56	0.46389	7 390	12 060	0 03568
16 000	31.3	979.69	0.38202	6 080	9 932	0.02900
14 000	34.8	1 211.04	0.30895	4 920	8 030	0.02292
12 000	38.8	1 505.44	0.24854	3 960	6 460	0.01828
10 000	43.9	1 927.21	0.19414	3 090	5 040	0.01305
8 000	50.0	2 500.00	0.14096	2 240	3 670	0.00900
6 000	59.6	3 552.16	0.10533	1 670	2 744	0.00406
4 000	74.4	5 535.36	0.06760	1 120	1 760	0.00112
2 000	100.7	10 140.00	0.03689	587	960	0.00091

TABLE No. 7a.—If a Solid 1-In. Square Cast-Iron Pillar is Loaded in Center with Weights Varying from 48 000 Lbs. (its Breaking Weight) to 24 000 Lbs., and if by Any of These Loads it were Bent to One-Half its Breaking Deflection on the Assumption that $\frac{S - W_1}{6\ W_1} = \delta_1$, then $S - W_1$ will Give a Radius Greater than $R_1 = 135.4$. Hence $X > S - W_1$, and there is a Horizontal Shifting of the Particles Composing the Pillar.

W_1	S.	$S - W_1$	$p\ (S - W_1)$	R.
48 000	72 000	24 000	14 600	140.3
44 000	66 000	22 000	13 475	172.7
40 000	60 000	20 000	12 250	219.5
36 000	54 000	18 000	11 025	286.8
32 000	48 000	16 000	9 800	381.1
28 000	42 000	14 000	8 575	514.9
24 000	36 000	12 000	7 300	664.8

TABLE No. 8.—Safe Loads of Round-Ended Solid 1-In. Square Cast-Iron Pillars Loaded in the Center. Maximum Strain = 2 670 Lbs. Tension, 16 000 Lbs. Compression. Given for all Pillar Lengths, both in Diameters and Radii of Gyration.

Lengths in diameters.	Lengths in radii of gyration.	Safe load.	Breaking weights.	Lengths in diameters.	Lengths in radii of gyration.	Safe load.	Breaking load.
6.0	20.7	15 820	.....	28.0	96.9	7 650	.....
6.2	21.5	15 750	72 000	28.8	99.8	7 450	14 000
6.7	23.2	15 660	65 000	29.0	100.4	7 400	.....
7.0	24.2	15 630	.....	30.0	102.9	7 100	.....
7.3	25.3	15 540	60 000	30.1	104.3	7 070	13 000
8.0	27.7	15 510	.....	31.0	107.3	6 800	.....
8.3	28.7	15 480	54 000	31.8	110.1	6 640	12 000
8.9	30.8	15 410	51 000	32.0	110.8	6 600	.....
9.0	31.1	15 400	.....	33.0	114.3	6 400	.....
9.5	32.9	15 300	48 000	33.2	115.0	6 360	11 000
9.7	33.5	15 260	47 000	34.0	117.7	6 209	.....
10.0	34.6	15 200	.....	35.0	121.2	5 920	10 000
10.1	35.0	15 150	45 000	36.0	124.7	5 640	.....
10.6	36.7	14 900	43 000	37.0	128.2	5 360	9 000
11.0	38.1	14 700	.....	38.0	131.6	5 080	.....
11.1	38.4	14 660	41 000	39.0	135.0	4 780	.....
11.3	39.1	14 580	40 000	39.4	136.4	4 500	8 000
11.6	40.2	14 460	39 000	40.0	138.5	4 280	.....
11.8	40.9	14 380	38 000	41.0	142.0	4 060	.....
12.0	41.5	14 300	.....	42.0	145.4	3 780	.....
12.7	44.0	13 950	36 000	42.3	146.5	3 600	7 000
13.0	45.0	13 800	.....	43.0	148.9	3 450	.....
13.5	46.7	13 510	34 000	44.0	152.4	3 300	.....
14.0	48.4	13 220	.....	45.0	155.8	3 150	.....
14.3	49.5	13 080	31 000	45.6	158.0	3 000	6 000
15.0	51.9	12 750	.....	46.0	159.3	2 800	.....
15.3	53.0	12 590	30 000	47.0	162.8	2 600	.....
15.8	54.7	12 580	29 000	48.0	166.2	2 400	.....
16.0	55.3	12 200	.....	49.0	169.7	2 200	.....
16.4	56.8	12 030	28 000	50.0	173.2	2 000	5 000
17.0	58.9	11 780	27 000				
17.6	61.0	11 440	26 000				
18.0	62.3	11 220	.....				
18.3	63.4	11 090	25 000				
19.0	65.8	10 800	.....				
19.4	67.2	10 620	24 000				
19.8	68.6	10 440	23 000				
20.0	69.2	10 350	.....				
20.6	71.3	10 080	22 000				
21.0	72.7	9 900	.....				
21.5	74.5	9 700	21 000				
22.0	76 2	9 500	.....				
22.4	77.6	9 380	20 000				
23.0	79.6	9 190	.....				
23.3	80.7	8 920	19 000				
24.0	83.1	8 800	.....				
24.3	84.2	8 710	18 000				
25.0	86.6	8 500	.....				
25.3	87.6	8 410	17 000				
26.0	90.0	8 200	.....				
26.4	91.4	8 080	16 000				
27.0	93.5	7 900	.....				
27.6	95.6	7 750	15 000				

To find the safe load (L_{1s}.) for eccentric loads. Let centric safe loads be L_s and eccentric safe loads L_{1s}.

Let centric breaking weights be W and eccentric breaking weights be W_1, then $L_1{}^s$: $L^s = W_1$: W and $L_1{}^s = \frac{L_s\ W_1}{W}$

TABLE No. 9.—Safe Loads of Round-Ended Solid 1-In. Square Wrought-Iron Pillars, Loaded in the Center. Maximum Strain = 12 000 Lbs. for all Pillar Lengths Given in Diameters and in Radii of Gyration. Formula for Eccentric Safe Loads in Table No. 8.

Length in diameters.	Lengths in radii of gyration.	Safe load.	Breaking weight.	Length in diameters.	Lengths in radii of gyration.	Safe loads.	Breaking weights.
10.0	34.6	11 810	.....	41.3	143.1	4 970	11 000
10.5	36.4	11 800	39 000	42.0	145.4	4 850	.....
11.0	38.1	11 800	.....	43.0	148.8	4 650	.....
11.6	40.2	11 800	37 000	43.9	152.0	4 470	10 000
12.0	41.5	11 790	.....	44.0	152.4	4 450	.....
13.0	45.0	11 750	.....	45.0	155.8	4 280	.....
13.5	46.8	11 700	34 000	46.0	159.3	4 110	.....
14.0	48.4	11 700	.....	47.0	162.8	3 950	9 000
15.0	51.9	11 600	.....	48.0	166.2	3 880	.....
15.6	54.0	11 550	31 000	49.0	169.7	3 650	.....
16.0	55.4	11 520	.....	50.0	173.2	3 500	.....
16.3	56.5	11 500	30 000	51.0	176.6	3 380	.....
17.0	58.8	11 400	.....	52.0	180.1	3 240	.....
17.1	59.2	11 390	29 000	53.0	183.5	3 100	.....
18.0	62.3	11 200	28 000	54.0	187.0	3 000	.....
18.7	64.8	11 110	27 000	54.5	188.8	2 950	7 000
19.0	65.8	10 900	.....	55.0	190.3	2 900	.....
19.6	67.7	10 720	26 000	56.0	193.8	2 770	.....
20.0	69.2	10 600	.....	57.0	197.4	2 650	.....
20.5	71.0	10 400	25 000	58.0	200.9	2 550	.....
21.0	72.7	10 200	.....	59.0	204.3	2 470	.....
21.4	74.1	10 060	.....	59.6	206.3	2 430	6 000
22.0	76.2	9 850	.....	60.0	207.8	2 400	.....
22.5	78.0	9 700	23 000	61.0	211.3	2 300	.....
23.0	79.6	9 550	.....	62.0	214.7	2 200	.....
23.5	81.7	9 250	22 000	63.0	218.2	2 120	.....
24.0	84.1	9 150	.....	64.0	221.7	2 070	.....
24.6	85.2	8 990	21 000	65.0	224.9	2 020	.....
25.0	87.6	8 880	.....	65.7	227.6	1 990	5 000
26.0	90.0	8 600	20 000	66.0	228.6	1 980	.....
27.0	93.5	8 350	.....	67.0	232.1	1 900	.....
27.1	93.7	8 320	19 000	68.0	235.6	1 850	.....
28.0	96.9	8 070	.....	69.0	239.0	1 800	.....
28.4	98.4	7 960	18 000	70.0	242.5	1 770	.....
29.0	100.4	7 800	.....	71.0	245.9	1 730	.....
29.7	102.7	7 600	17 000	72.0	249.4	1 700	.....
30.0	103.9	7 520	.....	73.0	252.9	1 650	.....
31.0	107.3	7 300	.....	74.0	256.3	1 600	.....
31.3	108.4	7 200	16 000	75.0	259.8	1 550	.....
32.0	110.8	7 000	.....	76.0	263.3	1 500	.....
33.0	114.3	6 780	15 000	77.0	266.7	1 450	.....
34.0	117.7	6 550	.....	78.0	270.2	1 400	.....
34.8	120.5	6 340	14 000	79.0	273.7	1 350	.....
35.0	121.2	6 290	.....	80.0	277.1	1 300	.....
36.0	124.7	6 090	.....	81.0	280.5	1 250	.....
36.7	127.1	5 900	13 000	82.0	284.0	1 200	.....
37.0	128.1	5 820	.....	83.0	284.0	1 250	.....
38.0	131.6	5 650	.....	84.0	290.9	1 100	.....
38.8	134.4	5 470	12 000	85.0	294.4	1 050	.....
39.0	135.0	5 420	.....	86.0	297.9	1 000	.....
40.0	138.5	5 210	.....	87.0	301.3	950	.....
41.0	142.0	5 020	.....	88.0	304.8	900	.....
.....	.....	.....	.....	89.0	308.2	850	.....
.....	.....	.....	.....	90.0	311.7	800	.....

TABLE No. 10.—COMPRESSION OF COMMON HARD BRICKWORK IN ACCORDANCE WITH EXPERIMENTS AT THE WATERTOWN ARSENAL. MODULUS OF ELASTICITY = 160,000 NEARLY.

Weights. Pounds per square inch.	Compression. In parts of unity.	Weights. Pounds per square inch.	Compression. In parts of unity.
50	.....	1 100	0.00680
100	0.00122	1 150	0.00722
150	0.00164	1 200	0.00764
200	0.00200	1 250	0.00783
250	0.00233	1 300	0.00813
300	0.00267	1 350	0.00855
350	0.00300	1 400	0.00892
400	0.00327	1 450	0.00900
450	0.00352	1 500	0.00928
500	0.00379	1 550	0.00946
550	0.00410	1 600	0.00960
600	0.00437	1 650	0.01000
650	0.00457	1 700	0.01013
700	0.00479	1 750	0.01031
750	0.00505	1 800	0.01056
800	0.00531	1 850	0.01108
850	0.00566	1 900	0.01116
900	0.00590	1 950	0.01147
950	0.00619	2 000	0.01171
1 000	0.00631	2 050	0.01204
1 050	0.00660	2 100	0.01238

TABLE No. 11.—PIERS OR WALLS OF COMMON HARD BRICK LAID IN BEST ROSENDALE CEMENT WITH 2 PARTS OF SAND, OR IN PORTLAND CEMENT WITH 3 PARTS OF SAND.—MAXIMUM STRAIN FOR SAFE LOADS, 300 LBS.

Breaking weight.	Length.	L^2	δ	Safe load.	δ_1
2 000	0.0	0.00	0.00000	300	0.00000
1 900	9.5	90.25	0.00868	297	0.00056
1 800	9.8	96.04	0.01847	295	0.00278
1 700	10.1	102.01	0.03000	290	0.00500
1 600	10.4	108.16	0.04160	285	0.00700
1 500	10.8	116.64	0.05600	280	0.01040
1 400	11.1	123.21	0.07124	275	0.01405
1 300	11.6	134.56	0.09000	270	0.01870
1 200	12.0	144.00	0.11080	260	0.02510
1 100	12.3	151.29	0.16000	250	0.03150
1 000	13.2	174.24	0.16666	240	0.04119
900	14.0	196.00	0.16666	225	0.04119
800	15.0	225.00	0.16666	200	0.04119
700	16.0	256.00	0.16666	175	0.04119
600	17.0	289.00	0.16666	150	0.04119
500	18.7	349.69	0.16666	125	0.04119
400	21.0	441.00	0.16666	100	0.04119
300	24.4	595.36	0.16666	75	0.04119
200	29.6	876.16	0.16666	50	0.04119
100	42.0	1 764.00	0.16666	25	0.04119

For sandstone safe loads may be increased by 0 — 40 per cent.
" granite " " " 60 — 160 "
" marble " " " 40 — 100 "
" limestone " " " 10 — 25 "

TABLE No. 12.—YELLOW PINE, SQUARE WOODEN POSTS, STRAIGHT GRAINED, PERFECTLY DRY AND LOADED IN THE CENTER.—WHEN NOT DRY, ONE-HALF THE SAFE LOAD TO BE TAKEN.—MAXIMUM STRAIN FOR SAFE LOADS, 800 LBS.

Length in diameters.	Breaking weight.	Deflection.	Safe load.	Deflection.	Safe load.	Safe load.
0	8 000	.....	800	.....	.....	.....
15	7 800	.00428	795	.0010	.....	.....
16	6 875	.02727	790	.0040	.....	.....
17	6 075	.05281	770	.0075	.....	.....
18	5 425	.07910	755	.0125	.....	.....
19	4 900	.10544	720	.0175	.....	.....
20	4 400	.13636	695	.0225	.....	.....
21	3 975	.16834	675	.0300	.....	.....
22	3 650	.20000	650	.0375	.....	.....
23	3 325	.21158	625	.0450	.....	.....
24	3 050	.27050	605	.0550	540	.....
25	2 825	.30531	580	.0650	460	.....
26	2 625	.34127	555	.0750	400	.....
27	2 400	.39000	530	.0850	320	.....
28	2 250	.42600	505	.0975	250	.....
29	2 075	.47590	475	.1100	220	460
30	1 950	.51710	460	.1225	190	375
35	1 400	.78571	350	.1975	90	150
40	1 100	1.0454	290	.2900	50	90
45	875	1.2810	240	.3900	30	60
50	700	1.7380	200	.4975	20	40
55	575	2.1521	165	.6150	10	30
60	500	2.5	135	.7300	5	20

DISCUSSION.

Mr. Thomson.

GEORGE H. THOMSON, M. Am. Soc. C. E.—The author states early in the paper that "it is reasonable to assume that at the intrados of the pillar at the point of greatest deflection a strain has been set up which is as great as the stress causing rupture in a specimen of the material, the height or vertical length of which specimen does not materially exceed its smallest diameter." While it may be reasonable to assume this, the validity of the assumption is questionable. The speaker did not care to deny it, because of the logical fact that from the denial of an antecedent nothing follows, but he believed that its negation was conceivable.

Mr. Berg.

WALTER G. BERG, M. Am. Soc. C. E.—Any attempt to obtain the true column formula should be welcomed by engineers, architects and designers, whether working in iron, steel, timber or other structural materials, and the author deserves, therefore, the thanks of the profession for his efforts to contribute to the subject. The deduction of a true column formula is one of the most complex problems in applied mechanics. The search for it is an attractive and alluring study, but it will probably prove as futile, in the sense of arriving at an absolutely final, theoretical solution of the question as the attempt to solve the perpetual-motion problem. When the fact is generally recognized that the subject is beyond the bounds of strictly theoretical investigations, then engineers will have advanced a great step in the matter. They will abandon deductive reasoning, and confine themselves to inductive methods. All their energies will be concentrated on physical tests as the basis of empirical formulas, which will become practically scientifically accurate as the number and value of reliable test results increase. This method is in perfect harmony with the general trend of column-formula investigations during the last century. Starting with the theoretical expressions of Euler and Tredgold, there has been a constant endeavor to extend and improve them by establishing coefficients to accord with additional test results as fast as obtained.

The author presents apparently theoretical formulas, but which, in fact, are premised on certain assumptions as to the physical behavior of the material, and subsequently supplemented by coefficients obtained from actual tests. The result is, therefore, a nominally theoretical solution dependent upon physical assumptions and results from practical experience. The value of the formulas presented can be best determined by comparing them with test results, especially of full-size pieces. It appears from Figs. 3 and 4 that the author's formulas for cast and wrought iron compare favorably with Gordon's formulas, and the results of a limited number of small-size tests made by Hodgkinson. This coincidence is not conclusive, be-

Mr. Berg. cause Gordon's formulas were based chiefly on Hodgkinson's work. It would be very interesting to see how the author's formulas would compare with the valuable test results of iron columns, especially of full-size specimens, obtained within recent years.

The ultimate breaking values for yellow pine columns, given by the author in Table No. 12, do not compare favorably with the plotted results of over 100 full-size tests of yellow pine columns and of empirical formulas established from these tests. It may be that the agreement in the case of iron would be better on account of the greater homogeneity of the material, but for yellow pine the author's formula fails to agree with actual facts, as clearly demonstrated by Fig. 11. This diagram shows that, as compared with test results and recognized empirical formulas, the author's values for yellow pine are too low for very long columns (which may be a good feature), and entirely too high for very short columns, while they are about correct for columns between 18 and 22 diameters.

The author's formula is $W_1 = 1\,740\,000 \div L^2$, where L is the length of the post in terms of the diameter and W_1 is a "weight less than the breaking weight." The values given in the second column of Table No. 12 under the heading "breaking weight" are, however, based apparently on the formula quoted. The curve in the diagram representing these values is hyperbolic in form. The author drops the formula arbitrarily at 15 diameters; if the curve is continued below 15 diameters, the unreliability of the formula from a practical point of view is shown by the fact that the breaking values for very short posts would be in excess of the crushing strength of short blocks of yellow pine. In other words the author's general form of equation

Breaking weight = a constant ÷ square of length

is not in accord with the results of tests, and differs radically from all other formulas known to the speaker, which give when plotted either a straight line or a curve of conchoidal shape.

In the following formulas, which are plotted on the diagram, W is the breaking weight in pounds per square inch.

The formulas of William H. Burr, M. Am. Soc. C. E., are based on tests at the Watertown Arsenal of full-size sticks of the average quality in practical use. They are:

$$W = 5\,800 - 70\,\frac{l}{d} \quad \text{for } \frac{l}{d} > 20.$$

$$W = 4\,400 \qquad \text{for } \frac{l}{d} < 20.$$

The formula proposed by James H. Stanwood, Assoc. M. Am. Soc. C. E., is based on the Watertown tests and on tests of full-size sticks made by Prof. G. Lanza. This formula is:

$$W = 4\,250 - 43\tfrac{1}{3}\,\frac{l}{d}$$

Mr. Berg.

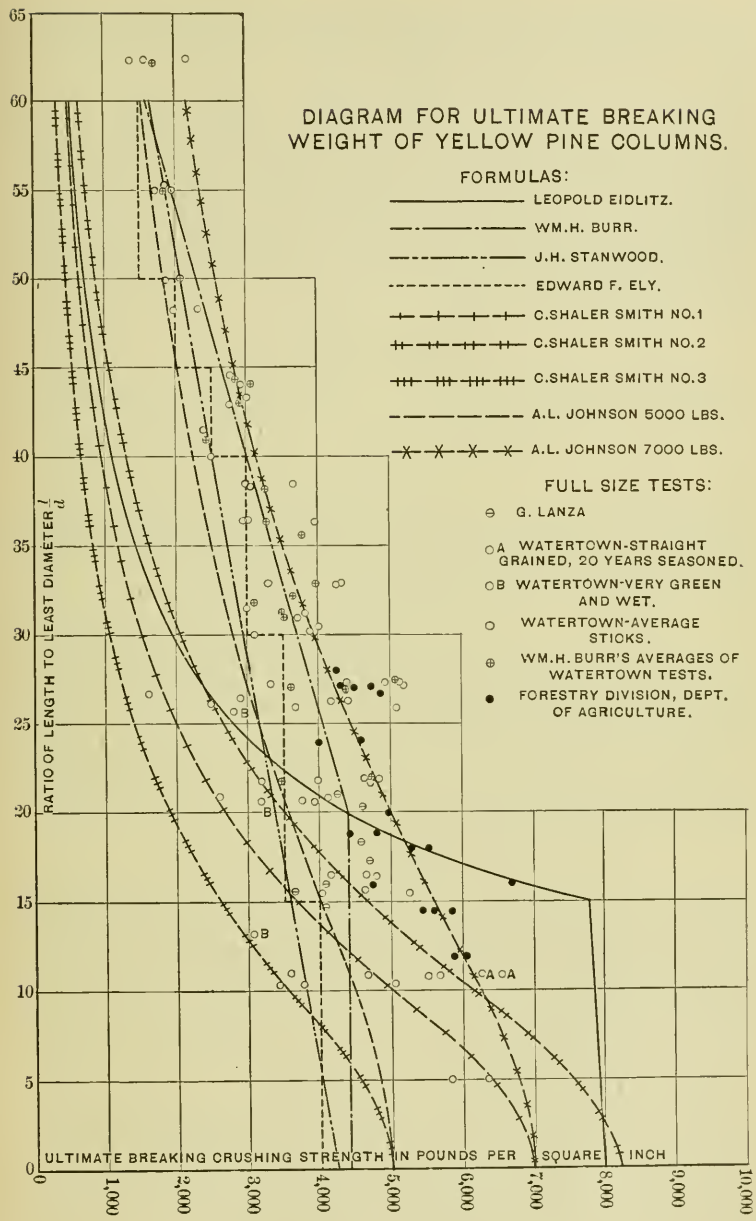

FIG. 11.

Mr. Berg. Prof. Edward F. Ely's rule based on the Watertown tests gives a series of averages between various limiting ratios of length to diameter. These averages are as follows;

Ratio	0–15	15–30	30–40	40–45	45–50	50–60
W.....	4 000	3 500	3 000	2 500	2 000	1 500

The formulas of the late C. Shaler Smith, M. Am. Soc. C. E., which are really Gordon's formula with the coefficients established from a series of over 1 200 tests of full-size sticks made by Mr. Smith in 1861–62, are plotted on the diagram as follows:

No. 1. For green, half-seasoned sticks, answering to the specification "good merchantable lumber."

$$W = \frac{5\,400}{1 + \frac{l^2}{250\,d^2}}$$

No. 2. For selected sticks, reasonably straight and air-seasoned under cover for two years and over.

$$W = \frac{8\,200}{1 + \frac{l^2}{300\,d^2}}$$

No. 3. For average sticks cut from lumber which had been in open-air service for four years and over.

$$W = \frac{5\,000}{1 + \frac{l^2}{250\,d^2}}$$

Mr. Smith recommended the third formula for general use, and it was employed in preparing the tables for yellow pine posts in "Trautwine's Engineers' Pocketbook."

The formula proposed by Mr. A. L. Johnson is based on tests of full-size sticks made by the Forestry Division of the United States Department of Agriculture; it is as follows:

$$W = F\left(\frac{700 + 15\,C}{700 + 15\,C + C^2}\right)$$

where $C = l \div d$ and F is the ultimate crushing strength of the material. The results of the tests on which the formula is based led Mr. Johnson to assume 7 000 lbs. per square inch as the value of F, and the diagram contains the curves of this formula, using both 7 000 and 5 000 lbs. as the unit base.

The tests of full-size yellow pine posts with flat ends plotted in the diagram are the following:

Prof. G. Lanza's tests of seven mill columns.

Tests made at the Watertown Arsenal of two straight-grained posts twenty years old, and of three very green and wet posts.

Prof. Burr's averages of these Watertown tests.

The average results of tests of about fifty posts made for the Forestry Division of the Department of Agriculture.

Mr. Prichard.

HENRY S. PRICHARD, M. Am. Soc. C. E.—The author takes the ground that "safe loads of pillars are not proportional to the breaking weights, but must be referred to a permissible maximum strain." To this view, which he holds in common with a number of recent authors, the speaker had no objection when it is confined to pillars so short that there is no danger of failure under any load which can be regarded as a reasonable possibility, but precisely because he agrees with the author that for columns in which bending occurs, the ratio between extreme fiber strains of different intensities is not the same as the ratio between the loads which produce them, he protested earnestly against making the intensity of the strain produced, instead of the relation of load to the breaking weight, the test of safety for long pillars. It is manifestly a mistake to speak of a load as safe when a small increase, something less than 25%, will cause failure, yet if a certain limiting intensity of strain, constant for all ratios of length to radius of gyration, is made the sole requirement, such loads will be allowed in many of the cases which occur in practice.

The author gives Tables Nos. 1 and 2 "derived from curves based upon the results of Hodgkinson's experiments," in which are given breaking weights from 1 000 to 96 000 lbs. for cast iron and from 500 to 52 000 lbs. for wrought iron, for pillars 1 in. square, and, under the heading "Compression due to W," also gives the distortions in terms of the lengths of the columns, corresponding to such weights, which show, as might be expected, that the distortions are relatively greater for large breaking weights than for small ones. From this he concludes that the distortions due to stresses of the same intensity as the breaking weights are the same as those he has tabulated for the breaking weights, and that therefore the distortions are relatively greater for large stresses than for small ones.

This conclusion is not a logical deduction from the tables cited. A pillar at the moment of failure has the stress distributed unequally, and the distortion therefore is caused by a varying stress whose mean intensity is the same as that of the direct compression, that is, the total load divided by the area. As in most cases some portion of the pillar is strained in excess of the elastic limit, and as the distortion for strain which is in excess of the elastic limit is known to be much greater relatively than for strain which is not, the natural inference is that distortion due to a varying stress whose mean intensity is the quotient of the total load divided by the area is greater than that due to a uniform stress whose intensity is the quotient of the total load divided by the area.

The Pencoyd experiments and those made at Watertown show that for wrought iron the modulus of elasticity is nearly constant through a range of stress from zero to the elastic limit. Among the Pencoyd experiments were some, made for the express purpose of determining

Mr. Prichard. the elasticity of wrought iron under compression, in which the specimens were secured laterally so that they could not bend. The results of one of these tests were given in a paper by James Christie, M. Am. Soc. C. E., entitled "Experiments on the Strength of Wrought Iron Struts."*

The author does not follow the same method in determining the breaking weight and the safe load, and the methods he does follow are not consistent with each other. The force of this objection may be understood by considering two metals having exactly the same elasticity, one of which would crush at a strain of 52 000 lbs. per square inch, and the other at a strain of 12 000 lbs. per square inch. The load which would cause failure in the pillar made of the weaker metal would cause a strain of 12 000 lbs. on the extreme fiber of the stronger metal, and would therefore be for the stronger metal what the author calls the safe load, and the way to obtain it would be to substitute S for C, and W_1 for W in his formula for breaking loads, assuming its accuracy for the sake of the argument, and adopting his notation.

Example.—Find the length of a 1-in. square pillar when the maximum extreme fiber strain is 12 000 lbs. per square inch and the load is 10 000.

$$\delta_1 = \frac{S - W_1}{6\ W_1} = \frac{12\ 000 - 10\ 000}{6 + 10\ 000} = \frac{1''}{30} \quad \ldots\ldots\ldots\ldots \quad (1)$$

$$R_1 = \frac{2 - (C_p\ (W_1 + p\ (S - W_1)) + C_p\ (W_1 - p\ (S - W_1)))}{2\ (C_p\ (W_1 + p\ (S - W_1)) - C_p\ (W_1 - p\ (S - W_1)))} \quad \ldots\ldots (2)$$

$$R_1 = \frac{2 - (0.000469 + 0.000351)}{2\ (0.000469 - 0.000351)} = 8471$$

$$L_1 = \sqrt{8\ \delta_1\ R_1} = \sqrt{8 \times \tfrac{1}{30} \times 8471} = 47.5 \text{ ins.} \quad \ldots\ldots\ldots\ldots (3)$$

The breaking weight for this length by Table No. 2 is between 8 000 and 9 000 lbs., which is less than the safe load. This result points inevitably to an error, which the speaker believed to lie in the author's system and not in the reasoning by which the result is deduced from it.

Although this method of obtaining the safe load is a logical deduction from the premises given by the author, and although equation (1) can be derived from $S = W_1\ (1 + 6\delta)$ given on page 372, equation (2) is given on page 386, and the same reasoning which develops $L^2 = 8\ R\ \delta$ on page 382, will develop equation (3), it is not the method he uses and the results are not what he obtains.

At one step in the train of reasoning, the author introduces the proportional equation $S - W_1 : C - W = W_1\ E : W\ E_1$, in which $S - W_1$ is written in place of X. If it is correct for any values of E

* See *Transactions*, Vol. xiii, p. 121.

and E_1 which different materials may have, it is correct for $E = E_1$, in which case it becomes: Mr. Prichard.

$S - W_1 : C - W = W_1 : W$, or $SW - WW_1 = CW_1 - WW_1$, or $SW = CW_1$. Hence the proportion, $S : C = W_1 : W$. This proportion is at variance with the author's remarks,* as follows: "and it appears that the relation of S to C can be as the respective weights W_1 to W only on the assumption that $1 + 6\,\delta_1 = 1 + 6\,\delta$, or that $\delta_1 = \delta$. It seems clear therefore that safe loads of pillars are not proportional to their breaking weights."

This discrepancy results from the use of different methods, not in accord, for determining the breaking weight and the safe load as defined by the author.

On the supposition of a varying modulus of elasticity, the formulas which the author develops from what he terms the breaking loads centrally applied, would have a different significance from what he assumes. To interpret them correctly the speaker considered first a pillar under the following conditions: Constant cross-section, homogeneous material, constant modulus of elasticity, and an axis coincident before deflection with the line of thrust.

Let M = the bending moment.
L = the length.
E = the modulus of elasticity.
I = the moment of inertia.
δ = the deflection.
W = the load which will just maintain the pillar in equilibrium when the deflection at the center is δ.
F = a factor which is constant for given conditions of loading and end connections.

The general equation for deflection is $\delta = \dfrac{M\,L^2}{F\,E\,I}$ ..... (4)

$$M = W\delta \qquad (5)$$

$$\delta = \frac{W\,\delta\,L^2}{F\,E\,I} \qquad (6)$$

$$W\,\delta = \delta\,\frac{F\,E\,I}{L^2} \text{ or } W = \frac{F\,E\,I}{L^2} \qquad (7)$$

This is Euler's equation, and as he has shown that for a column with hinged ends $F = \pi^2$, it can be written

$$W = \frac{\pi^2\,E\,I}{L^2} \qquad (8)$$

Let W_1 be any load less than W,
W_2 " " greater than W.

It is apparent from equations (6) and (7):

* See page 372.

Mr. Prichard *First.*—The load W will maintain the column in equilibrium under any deflection.

Second.—If W_1 is substituted for W the deflection will become zero.

Third.—If W_2 is substituted for W the deflection will increase until the elastic limit is reached, which is the beginning of failure.

If E should grow less as the strain increased, in the ratio assumed by the author, it would not affect the accuracy of the deduction that any load $W_1 < \frac{\pi^2 E I}{L^2}$ would produce no deflection, but it would affect the deduction that W would maintain the pillar in equilibrium for any deflection, as the bending moment would no longer be simply $W \delta$, but would be increased by the shifting of the axis caused by the different values of E on the concave and convex sides of the pillar.

Under these conditions a smaller load than that required to start deflection will maintain the pillar in equilibrium for a given deflection, if such deflection has already been produced, and it is for these loads under the supposition of a varying modulus of elasticity that the author's formulas deduce, approximately, the deflections and lengths.

The difference between the actual conditions met with in practice and the ideal ones assumed as the basis of formulas (6), (7) and (8) has a marked influence on the behavior of pillars, and this fact should be considered in proportioning them.

CORRESPONDENCE.

Mr. Morison. George S. Morison, Past-President Am. Soc. C. E.—The paper is valuable for its suggestions rather than for its conclusions. The assumption that failure always occurs because some portion of the material in a pillar is strained beyond its ultimate strength is not correct. Neither is the assumption that the extreme local strain is reached by a combination of direct compressive strains and bending strains always correct. The destruction of compression members is occasioned by a great variety of causes, in which the distortion of material when strained beyond the elastic limit usually plays a greater part than the ultimate strength of the metal. A pillar will be crippled when some portion of its material has been strained beyond the elastic limit, although nothing is broken; but a subsequent pressure considerably less than that which crippled the pillar may increase the crippling so much that actual destruction of metal will occur. Pillars also fail through the buckling of individual parts, and this is the usual source of failure when very thin sections are used.

The class of pillars to which the author has applied his analysis are of so simple section that some of these disturbances are eliminated;

still it is not correct to assume that the pillars bend in a single curve from end to end; the curve is more likely to have two points of contrary flexure and the result would be that the strain which produces rupture and which is determined by the radius of curvature, would occur with a very much less lateral deflection than the author's analysis assumes. Mr. Morison.

The formulas which are the basis of the analysis are empirical formulas based on the result of actual tests. The author takes these as accurate measures of the strength of pillars of dimensions corresponding to the assumed conditions, and calculates how much these pillars must deflect laterally to produce a strain in extreme fibers equal to that required to break the material, this being a tensile or a compressive strain according to the length of pillar. He then decides that the factor of safety should not be applied on the load placed upon the pillar, but on the strain in extreme fibers and calculates what pressure on the ends of the pillar would correspond to the working strain which his factor of safety permits on extreme fibers of metal. His demonstration shows correctly that the longer the pillar, the larger the relative amount of strain due to lateral deflection or bending, and, as his factor of safety is applied upon a resultant of which this strain forms a part, it follows that the factor corresponding to the division of the breaking strength of the whole pillar by the safe load on the whole pillar, which is the usual factor of safety, will be less than the factor corresponding to the breaking strain in extreme fibers divided by the working strain in extreme fibers, which is the author's factor of safety. The result is that as the length of pillar increases, the general factor of safety decreases, the fiber factor remaining constant.

While this may be theoretically correct, it can hardly be considered good practice. It overlooks the fact that a factor of safety is intended to cover a great many uncertainties and irregularities, and the longer a pillar becomes, the greater the opportunities for these irregularities. When a pillar passes a certain length it is likely to break by tension on the convex side, and even safe working loads may produce tension on one side and compression on the other. Furthermore, comparatively small disturbances may reverse the deflection, and the pillar which, under one loading, deflects to the right, may under another, which is equally likely to occur, deflect to the left; the result is that, although the actual strains may never exceed what the author has selected as safe working strains on extreme fibers, they may approach this amount, both as positive and negative strains, and the variation between extremes may be double the maximum in either direction. As it is generally conceded that the measure of variation in strains is at least as important as the measure of maximum strains, it follows that what would be a perfectly safe working strain for a short pillar under compression would be absolutely dangerous in pillars subject to such

Mr. Morison. reversals. Again, the longer the pillar, the greater the liability to distortion from accidental blows; a blow which would be entirely harmless on a pillar 10 ft. long would exert nine times as much effect on a pillar 30 ft. long; and if a load came on before the effect of this blow was removed, disaster might follow.

The paper is perhaps an illustration of the dangers of applying theoretical analysis to empirical formulas based on tests which the analyzer has not himself observed. It might be instructive to make tests of pillars of the precise dimensions which have been considered, and observe how far the action of the pillars under strain corresponds with the changes of shape which have been assumed as the basis of his analysis.

The paper, however, brings out one very important feature, which seems in danger of being neglected in architectural construction, though engineers have long recognized its absolute importance. It is briefly the fact that a pillar is not simply a compression member, but a beam as well, and that no pillar is properly designed in which its duties as a beam are not considered. The fundamental requirement of a beam is that it must be able to resist flexure, and this resistance causes bending strains in the parts on opposite sides of the neutral axis. In order to transmit these strains from the point of application to the flanges and from one flange to the other, it is absolutely necessary that the two sides should be connected by something in the nature of a continuous web. In the case of a solid beam this is simple enough. In the case of a built beam, like a plate girder, it requires care in the design; the web must be continuous, and the rivets connecting the web with the flanges must always be enough to transmit to the flanges all the strains which those flanges may have to carry. If an open web is used it must be of some continuous pattern, so that at every point of application no strain but a longitudinal one is carried by the flanges the shearing strain being taken entirely by the web members. In other words, no pillar is properly designed unless it has a web which forms a continuous bracing like the web of a girder or truss.

If cast-iron pillars are used, and the general opinion of engineers would certainly be against using them at all except for short sections, the best form is undoubtedly the round hollow column, provided it be cast vertically; but whatever form is used, it should be of continuous section.

The forms of wrought-iron pillars which are generally approved may be divided into box and lattice columns. The box column is undoubtedly the strongest, and, except for the difficulty of inspection, the best; whether the Phœnix form be used or the square column, made of two plates and two channels, the result is a column which meets all theoretical requirements and gives excellent results. Latticed columns are all right, provided the lacing is properly proportioned and is continuous; they are now the favorite column in

bridge work, partly for convenience of connections, which are generally in parallel planes, and partly because of accessibility for cleaning and painting; on the other hand, they are less convenient, and comparatively little used in architectural work. Mr. Morison.

Another class of pillars has been made of an H form which meets the necessary requirements. These may be made entirely of channels, entirely of I beams, of a combination of channels and I beams, and of late they have been made very largely of a single web plate and four Z irons. All of these pillars have continuous webs and are well adapted to resist bending; care must be taken that the flanges are thick enough to resist buckling, and the tendency to buckle is very greatly reduced by supporting the flanges at moderate intervals.

Varieties of pillars have been introduced from time to time, but generally discarded before very long, which are without continuous webs, being composed of several longitudinal members connected at intervals only. For short pillars under moderate strains, and especially in those places where there is no tendency to any transverse strain, this form of pillar may be admissible, but even then it cannot be economical, as the transverse unit is a dimension of one of the sections of which the pillar is composed and not the width of the pillar itself. To show the weakness of such a pillar it is only necessary to analyze it in the way in which the writer analyzed his cast-iron pillars; to calculate what bending strain must exist in the flanges of a properly designed pillar of a similar cross-section when the pillar fails and then calculate the web required, both in dimensions and connections, to produce these bending strains in the flanges. A defective pillar of this class, which, unfortunately, is now being quite extensively used, may be cited; it is formed of eight angles, arranged as shown in Fig. 12, and connected at intervals by short bent plates. This pillar is very convenient for connections, and, if the connecting plates were continuous, would be a good pillar. As constructed there is no provision for transmission of bending strains except by the transverse stiffness of the angles, and the pillar must necessarily fail under comparatively small bending strains. It is everywhere an inferior pillar, and if used where its transverse stiffness is relied upon to resist wind pressure, it may be classed as dangerous.

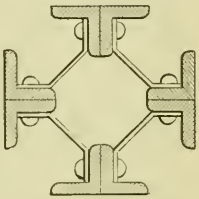

FIG. 12

Passing to another feature, it will be observed that Table No. 8, safe loads of cast-iron pillars, is based on maximum strains of 16 000 lbs. to the inch, while Table No. 9, safe loads of wrought-iron pillars, is based on maximum strains of 12 000 lbs. to the inch. Wrought iron is practically no longer used; but in columns of such length that flexure plays any considerable part in the working strains it certainly is not right to subject cast iron to any greater strains than are permissible on wrought iron.

Mr. Eidlitz. LEOPOLD EIDLITZ, Esq.—It is probable that strain at the middle of the intrados of a pillar at the point of breaking by flexure is greater than the stress causing rupture in a specimen of material when tested to ascertain its resistance to compression, for the reason that the ultimate strain in the pillar is produced in a mathematical point, which is equivalent to the compression of an infinitely thin plate, where all bending strains are eliminated. It is not probable that the ultimate strain of a bent pillar is less than the tested resistance to compression of the material.

For the purpose of analysis, it is therefore "reasonable to assume that at the intrados of a pillar at the point of greatest deflection a strain has been set up which is as great as the stress causing rupture in a cube of the material in question." The author would add that probable errors resulting from this assumption tend to place the results on the safe side.

Beams of a given length and of uniform elasticity when bent under transverse loads deflect proportionately to the load. It is immaterial, therefore, whether in order to ascertain the value of safe loads an assumed factor of safety is applied to the breaking weight of the material, or to its ultimate extreme fiber strain. This is not the case in the bending of a pillar. Given fractions of breaking weight result in variable maximum fiber strains, hence in strength of resistance disproportionate to loads. This condition is enhanced by the fact that matter is not uniformly elastic.

An accurate working formula which includes the results of bending under eccentric loads as differing in its deflection from that caused by centric loads seems desirable, because values under small eccentricities are in practice underated, hence frequently entirely neglected as being covered by the factor of safety; therefore it is stated on page 371.

"The questions examined in this paper are mainly what is the maximum strain in pillars compressed endwise in the center of resistance by loads less than the breaking weights, or by loads not applied in the center of resistance."

In order to ascertain the maximum strains of weights less than the breaking weight, it is necessary to know the resistance at the point of breaking, and inasmuch as the resistance to the breaking weight is the total potential energy of the pillar, after it is compressed vertically by the breaking weight, the breaking weight must first be ascertained in order to know that resistance.

There are two methods pursued to ascertain the breaking weights of pillars of various lengths, neither of which, however, contemplates the use of a coefficient, as is erroneously suggested in the discussion. The method proposed to ascertain breaking weights of various pillar lengths in accordance with Tables Nos. 3 and 4 has been as follows:

For Cast Iron.—Ten diagrams of resistance at the center of the pillar were plotted by the method pursued in Fig. 1. They varied from

compression of 72 000 lbs. at the extrados and 96 000 lbs. at the intrados to nil at the extrados and 96 000 lbs. at the intrados; thence from 10 000 lbs. tension at the extrados and 96 000 lbs. compresssion at the intrados to 16 000 lbs. tension at the extrados and 24 000 lbs. compression at the intrados. Mr. Eidlitz.

For Wrought Iron.—Ten diagrams of resistance at the center of the pillar were plotted, varying from 42 400 lbs. compression at the extrados, and 52 000 lbs. compression at the intrados, to 42 000 lbs. tension at the extrados and 52 000 lbs. compression at the intrados.

From these diagrams the deflection, breaking weights, radii and lengths were computed, and Tables Nos. 3 and 4 formed from curves connecting the various figures of deflection, breaking weight, radius and length.

Tables Nos. 1 and 2 were formed by computing from each breaking weight the deflection, radius and pillar length by the process and formulas stated in the paper. In relation to this matter, attention is drawn to the following paragraph in the paper:

"Under conditions of uniform elasticity, this value of δ is absolutely correct; but inasmuch as elasticity is variable, it would be true only if the horizontal sectional planes drawn transversely to the pillar length were to be curved during the process of bending," which of course is not the case.

When the results of Tables Nos. 1, 2, 3 and 4 were plotted in Figs. 3 and 4, it was found that the curves of Tables No. 1 and 2 agreed as nearly as can be expected with Hodgkinson's experiments (variation of the material being allowed for). Tables Nos. 3 and 4 showed breaking weights somewhat larger for the same pillar lengths, and although those results are doubtless perfectly correct, Tables Nos. 1 and 2 were accepted as the basis of further analysis and calculation, by reason of their being in accord with actual experiments, and for the further reason that the breaking weights are on the safe side.

An examination of Figs. 3 and 4 will show plainly that neither of these curves agree with the curve made in accordance with Gordon's formula. Gordon's curve becomes materially inaccurate in pillar lengths less than 28 to 30 diameters. This is fully in accord with Prof. Rankine's statement* that Gordon's formula gives approximate results only for long rods or pillars.

In the interest of a preference for empirical formulas, which in the language of the discussion "will become practically scientifically accurate as the number and value of reliable test values increase," and in the interest further of separating imperfections of material (in its nature and also in form and magnitude as used in structural parts), also imperfections of methods of testing, from the fundamental cause of bending, which is purely resistance to compression and tension, elasticity in fact, Gordon's and other kindred formulas may be cor-

* See Rankine's "Civil Engineering," p. 237.

Mr. Eidlitz. rected as follows: Gordon's formula gives approximate values for breaking weights of long pillars (of 30 diameters and upwards). It is not available for pillars less than 30 diameters in length. The main cause of this imperfection is to be found in the fact that his coefficient, a, as given by Rankine, is not flexible. It is here intended to show how by the definitions of the deflection δ, the radius R, and the pillar length L, which form the basis of Tables Nos. 1 and 2, the coefficient a may be made flexible; the formula then will cover all pillar lengths with sufficient accuracy for practical use. The error will be less than $1\frac{1}{2}$ per cent.

Gordon's and other kindred empirical formulas are all based upon the equation $\delta\ W = \frac{d^3}{6}(C - W)$ (where δ is the deflection at the breaking point, W the breaking load, and C the ultimate resistance to crushing of the material) rendered in the form $W = \frac{C}{1 + 6\,\delta}$ when the diameter $d = 1$. This value of W is not true, however, in the case of cast-iron pillars of lengths greater than 12 diameters. For these the formula is $W = \frac{T}{6\delta - 1}$ (where T is the ultimate resistance of the material to tension). This fact seems to have been overlooked.

The next effort in composing a working formula has been to express the value of $6\,\delta$, by some $a\,L$*, or $a\,L^2$, where a is a fixed coefficient of L or L^2, and it was assumed that $a\,L$ or $a\,L^2$ may be made to approximate the value of $6\,\delta$ with sufficient accuracy to express the value of W, for long pillars at least.

Now, $\delta = \frac{L^2}{8\,R}$; hence $a = \frac{6}{8\,R}$. The radii of cast-iron pillars between 8 and 50 diameters in length vary between 67 and 460 diameters; consequently in a pillar 8 diameters long with single flexure, $a = \frac{6}{8 \times 67}$ and in a pillar 50 diameters long $a = \frac{6}{8 \times 460}$ (the radii of wrought-iron pillars for 10 to 50 pillar lengths vary between 250 and 327). This shows that to be accurate a must be variable.

It may be observed, however, that R becomes greater or less at some ratio which contains a function of the pillar length, and it may be possible, therefore, to express $\frac{6}{8\,R}$ by $\frac{1}{\pm n\,L \pm c}$ where c and n are constants for each material. Hence, a formula—

$$W = \frac{C}{1 + \frac{L^2}{\pm n\,L \pm c}} \quad \text{or} \quad W = \frac{T}{\frac{L^2}{\pm n\,L \pm c} - 1}$$

may be made sufficiently accurate (within certain given lengths of pillars, at least) of a given material. Upon this principle the following

* L or L^2 may be considered as expressed in diameters or in radii of gyration, then a varies accordingly.

formulas for breaking loads of cast and wrought iron have been computed: Mr. Eidlitz.

For breaking weights, W, for cast-iron pillars from 5 to 11 diameters in length $W = \dfrac{C}{1 + \dfrac{L^2}{245 - 14\,L}}$

where $C = 96\,000$ lbs.

For breaking weights, W, for cast-iron pillars 11 to 30 diameters long $W = \dfrac{T}{\dfrac{L^2}{15.7\,L - 86} - 1}$

where $T = 16\,000$ lbs.

For breaking weights, W, for cast-iron pillars 30 to 50 diameters long $W = \dfrac{T}{\dfrac{L^2}{10\,L + 87} - 1}$

For wrought-iron pillars from 8 to 50 diameters long,

$$W = \frac{C}{1 + \dfrac{L^2}{310 + 2.5\,L}}$$

For compound flexure a must be divided by 4, inasmuch as L^2 becomes $\left(\dfrac{L}{2}\right)^2$

From the text of the paper it appears clear that outside of the seven formulas enumerated in the summary*, all factors of safety or statements of moduli of elasticity are not an outcome of its deductive demonstrations, but merely assumptions, currently accepted, for the convenience of illustration.

It is well known that yellow pine is not uniformly elastic, and it needs, therefore, no diagram nor extended enumeration of breaking weights and the formulas by which they are computed from the results of various experiments to show that they differ from those resulting from an assumed uniform elasticity which does not exist, but which has in this case been assumed for the sake of brevity in computing safe loads.

When in a structural part the maximum strain is directly proportionate to the weight, it is immaterial whether a determined factor of safety is applied to either; but when, as in the case of pillars, this is not the case, the factor of safety must be applied to the determination of the permissible maximum strain. If it is probable, or even possible, that the computed actual loads may be exceeded by 25% or any other quantity whatever, that quantity must be added to the primarily computed load, or if in the mind of the engineer a doubt still exists as to the safety of an unusually long pillar (and it must be remembered

* See page 397.

Mr. Eidlitz. here that in construction as now practiced, pillars are invariably short, not long, viz., exceeding 20 diameters), or if he is influenced by habit or by a laudable desire to limit deflection to a minute imperceptible quantity, he may in such a special case increase the factor of safety.

Referring again to the quotation on page 423, and applying it to the problematical case of a pillar of the elasticity of wrought iron and an ultimate strength of 12 000 lbs., the method of computing the pillar length must be in accordance with a condition of uniform elasticity as follows:

$$E = \frac{W}{C_p} = \frac{12\,000}{0.0005} = 24\,000\,000$$

$$R = \frac{E}{2\,p\,(C - W)},\ \delta = \frac{C - W}{6\,W}$$

$$L^2 = \frac{8\,E}{12\,p\,W} = \frac{1.08843 \times 24\,000\,000}{W} = \frac{26\,122\,000}{W},$$

W being 10 000 lbs. Hence, $L^2 = 2\,612.2$ and $L = 51.1$. According to Table No. 4, which is accurate, the length of the problematical pillar which breaks under 10 000 lbs. is 50.9.

Pages 384 to 387 of the paper, containing a section on "the deflection and radius caused by $W_1 < W$ when placed in the center of resistance," and also Table No. 7a prove conclusively, as stated in the heading of Table No. 7a, that X is greater than $S - W_1$, which means distinctly that $X = S + n - W_1$, when n is a function of W_1.

The discussion contains the sentence: "At one step in the train of reasoning the author introduces the proportional equation $S - W : C - W = W_1\ E : W\,E_1$ in which $S - W_1$ is written in place of X" (evidently by the speaker).

Inasmuch as the equation alluded to in the text of the paper contains the term X and not the term $S - W_1$, this proves the speaker's deductions following the substitution of $S - W_1$ for X to be erroneous.

The formula for the resistance to breaking when W_1 is less than W, $X = (C - W)\,\frac{L^2}{L_1{}^2}$, is proved beyond controversy by the fact that when the radius is computed from the value of X, and when the deflection is computed by either of the formulas No. 7*, then $8\,R\,\delta = L^2$, while for any other value of X this is not the case.

Hodgkinson, speaking of Euler's theory of the strength of pillars, says:

"It appeared from the researches of this great analyst, that a pillar of any given dimensions and description of material required a certain weight to bend it, even in the slightest degree; and with less than this weight it would not be bent at all (Acad. de Berlin, 1757). Lagrange, in an elaborate essay in the same work, arrives at the same conclusion. The theory, as deduced from this conclusion, is very

* See page 397.

beautiful, and Poisson's exposition of it in his 'Mecanique,' 2d Edition, Vol. 1, will repay the labor of a perusal. Mr. Eidlitz.

"I have many times sought, experimentally, with great care for the weight producing incipient flexure, according to the theory of Euler, but have hitherto been unsuccessful. So far as I can see, flexure commences with weights far below those with which pillars are usually loaded in practice. It seems to be produced by weights much smaller than are sufficient to render it capable of being measured. I am therefore doubtful whether such a fixed point will ever be obtained, if indeed it exist. With respect to the conclusions of some writers, that flexure does not take place with less than about half the breaking weight; this, I conceive, could only mean large and palpable flexure; and it is not improbable that the writers were in some degree deceived from their having generally used specimens thicker, compared with their length, than have been usually employed in the present effort."

The author would add that he cannot conceive how Euler's formula throws any light upon the maximum strain in pillars compressed endwise in the center of resistance by weights less than the breaking weight, or by weights not applied in the center of resistance.

The discussion contains a considerable amount of matter which has no direct bearing upon the subject of maximum strain under various loads and differing conditions of loading of pillars, which on that account requires no reply on the author's part, excepting inasmuch as it indirectly affects principles involved.

Mr. Morison says: "The assumption that failure always occurs because some portion of the material in the pillar is strained beyond its ultimate strength is not correct." How can matter fail unless strained beyond its strength?

The same writer says also: "It is not correct to assume that pillars bend in a single curve from end to end." They always bend in this way when rounded at the ends as stated in the heading of the tables.

He says also: "The formulas which are the basis of the analysis are empirical formulas based on the results of actual tests." There are no formulas, empirical or otherwise, at the basis of the analysis. The analysis defines deflection and radius by geometrical and mathematical deduction. In another place the same author says: "The paper is perhaps an illustration of the dangers of applying theoretical analysis to empirical formulas based on tests which the analyser has not himself observed." Analysis tends to correct empirical coefficients in the interest of accuracy, a process which cannot be conceived to involve danger to science. The idea that analysis necessarily involves personal observation of tests quoted is not held by the author.

Another criticism by the same writer is:

"Passing to another feature, it will be observed that Table No. 8, safe loads of cast-iron pillars, is based on maximum strains of 16 000 lbs. to the inch, while Table No. 9, safe loads of wrought-iron pillars,

Mr. Eidlitz. is based on maximum strains of 12 000 lbs. to the inch. Wrought iron is practically no longer used; but in columns of such length that flexure plays any considerable part in the working strains it certainly is not right to subject cast iron to any greater strains than are permissible on wrought iron."

Table No. 8 contemplates a permissible maximum tensile strain for cast iron of 2 670 lbs. only.

Engineering handbooks published by various rolling mills and others having the authority of competent engineers compute pillars planed top and bottom or pillars of continuous length held in position by girders and beams abutting upon them and fastened to them with bolts or rivets at every story as subject to compound flexure under a safe load.

Deflections of pillars usually employed in building are exceedingly small under safe loads; for instance deflections of wrought-iron pillars from 12 to 20 diameters long (under loads varying from 11 790 to 10 600 lbs.) are 0.003 to 0.022 diameter.

The movement of the pillar head from a horizontal position to one sufficiently inclined to correspond with the stated deflections is so small that the strains generated are inoperative, because the movement is abundantly practicable within the limits of inaccuracy of construction such as exists in practical building.

If bending were continued to the breaking point, then, no doubt, compound flexure would ensue, but in the absence of loads greater than safe loads, pillars bend with single flexure.

It is also a well-known fact that eccentric loading is under-rated in the absence of a working formula which by one process gives eccentric breaking loads as compared with centric breaking weights and the strength of material.

These considerations have resulted in the analysis of the strength of pillars, and go to show that safe loads are governed by maximum strain, and not by breaking weights, or else many buildings constructed under the old system would show more serious defects than have been discovered as yet, and also that with a table of safe loads at the command of the engineer or architect, no eccentric load, no matter how small, should be neglected on the plea that the factor of safety being applied to weights instead of strains covers a multitude of defects not critically examined.

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TRANSACTIONS.

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No. 779.

THE TWENTY-EIGHTH STREET CENTRAL STATION OF THE UNITED ELECTRIC LIGHT AND POWER COMPANY.

By H. W. YORK, Jun. Am. Soc. C. E.

PRESENTED MARCH 18TH, 1896.

WITH DISCUSSION.

The history of the development of the electrical industries for the past few years probably presents the most remarkable example of rapid growth ever recorded, and nowhere is this better exemplified than in the central stations of this and other cities, where the current is generated for distribution over areas of greater or less extent for the supply of light and power. In isolated plants the apparatus is generally small and simple in construction, and the generation of electricity is a matter of secondary importance ; the cost of the current generated is comparatively immaterial, and a plant once installed runs until it refuses absolutely to do any more work, when it is sold for junk and another put in its place. In central stations, however, this is not the case. Here the cost of the current generated is a matter of the first importance, and as the growth of business calls constantly for additional capacity, and competition demands constantly better service

and cheaper methods of generation and distribution, in such stations will be found the results of the latest discoveries, inventions and improvements, affecting the entire apparatus required for the generation of current, from the boilers to the switchboard. Here everything is being constantly revolutionized ; what was good practice yesterday is obsolete to-day, and what a few years ago was looked upon with wonder and admiration is laughed at to-day as being old-fashioned and out of date.

Boiler-making has been almost entirely revolutionized to meet the requirements of the electrical business ; engine-building has undergone changes even more remarkable, but it is in the strictly electrical apparatus that the greatest strides have been made, the other industries simply following in the wake as rapidly as possible and trying to meet the ever-changing conditions. In dynamo-building one of the most noticeable changes to the layman is the increased size of the units. A few years ago a 600-light incandescent dynamo was considered a monstrosity, and when the first were built wiseacres predicted that they could never be made to operate successfully, that they were too big and awkward to handle, and that it was carrying too many eggs in one basket to have 600 lights on one dynamo. To-day a 6 000-light dynamo is considered little more than a baby, while a 600-light machine is scarcely large enough to excite the field of a good-size alternator.

With the increase in the size of the dynamo there has naturally come a reduction in the speed of the armature, making it possible to connect the armature shaft directly to the engine shaft, either by a rigid or a flexible coupling, and to do away with belts, which, under the conditions and at the speeds—seldom less than 6 000 ft. per minute—required in most electrical work are a source of never-ending trouble ; this is certainly a long step in the right direction. The author does not wish to incur the enmity of belt men by running down their goods. Belts are very good things and all right in their place, but an electric station is no place for them.

With these changes have come naturally an increased efficiency and various other improvements which, though fully as important as those mentioned, are less noticeable to a person unfamiliar with electrical matters.

The author has been connected for a number of years with the United Electric Light and Power Company, a corporation supplying

electricity for all purposes in the city of New York and formed by the combination of interests of the oldest companies in the field. The stations which the new company acquired and attempted to operate had all been doing service for several years, and the apparatus was naturally somewhat antiquated.

After making numerous changes and improvements, both in the methods of generation and distribution of current, there came a time when alterations would no longer answer the purpose, and the author was instructed to design and erect a new station, to take the place of all the old ones and embody, as far as possible, the best modern practice for the economical generation and distribution of current, as well as the most efficient and reliable service.

The first portion of this station has been erected, and the apparatus has been installed and is in operation, and it is intended to give in this paper a brief description of it as an instance of modern practice in this particular line.

When the subject of a new station was taken up, one of the stations operated by the company was on the south side of East Twenty-ninth Street, the property extending west 280 ft. from the East River and including also eight lots directly in the rear of the station on the north side of East Twenty-eighth Street, but having no water front. This Twenty-eighth Street property was vacant, with the exception of a few small one-story buildings, and was used as a storage yard. This was considered a good central location for the district to be covered, from the Battery to Fifty-ninth Street, and it was decided to design the station to run from street to street, and to erect and equip first the Twenty-eighth Street half, which would have sufficient capacity to take care of the load of all of the present stations, and then to tear down the old station and continue the new one through to Twenty-ninth Street, to provide for future growth of the business. The Twenty-eighth Street half is now erected, and sufficient apparatus has been installed to care for all the load, with the exception of that carried by the old Twenty-ninth Street plant. It is the intention to install during the coming summer the remainder of the machinery in the half already erected, and then remove the Twenty-ninth Street station and complete the new one.

One of the first considerations in laying out work in a large city where property is so valuable as in New York is economy of space, and

in this connection the author considers it worthy of notice that in this station 20 000 H. P. of engines, together with the boilers, pumps, heaters, condensing apparatus, dynamos and switchboard, and storage for 6 000 tons of coal, are all on a plot of ground 160 ft. 11 ins. by 197 ft. 6 ins. All machinery, including the boilers, is on the ground floor, and yet there is plenty of light, air and ample space for working around all the apparatus, both for its ordinary operation and for making repairs.

The building already erected has a frontage of 160 ft. 11 ins., and is a steel frame structure with a brick filling in the walls, except on the north end, which is covered by a corrugated iron curtain wall which will be removed when the Twenty-ninth Street side is built. Figs. 1, 2 and 3 give a general idea of the construction of the building. Fig. 1 is a cross-section of the entire structure, Fig. 2 a longitudinal section through boiler room, and Fig. 3 the roof plan; Fig. 4 shows the front elevation of the building as it appears from Twenty-eighth Street. This entire front wall is hollow and is carried up above the roof, to prevent the noise from the machinery annoying the patients in Bellevue Hospital, which is directly across the street. The wall is really composed of two walls, one 12 ins. thick on the outside, and one 8 ins. thick on the inside, with a 2-in. air space between them. These two walls are bonded together at every sixth course vertically by bricks spaced 20 ins. horizontally, as shown in detail in Fig. 5.

Double windows are also placed on this side of the building, to prevent the noise of the machinery from causing annoyance, and this arrangement has been so successful that, standing directly in front of the building, it is impossible to tell whether or not the machinery is in operation. The foundations for all the machinery rest on solid rock, as do the foundations for the building itself, with the exception of one small corner which has no great weight to carry.

To prevent the transmission of any possible vibration to the adjoining buildings, the foundation walls were kept 1 in. away, and the space filled with sand up to the surface of the ground. Above this point the adjoining walls were covered with tarred paper and the brickwork built against this, thus preventing any possible bond between the brickwork of the station and that of the adjoining buildings. As a matter of fact these precautions were unnecessary as the station walls themselves are absolutely free from any perceptible vibration.

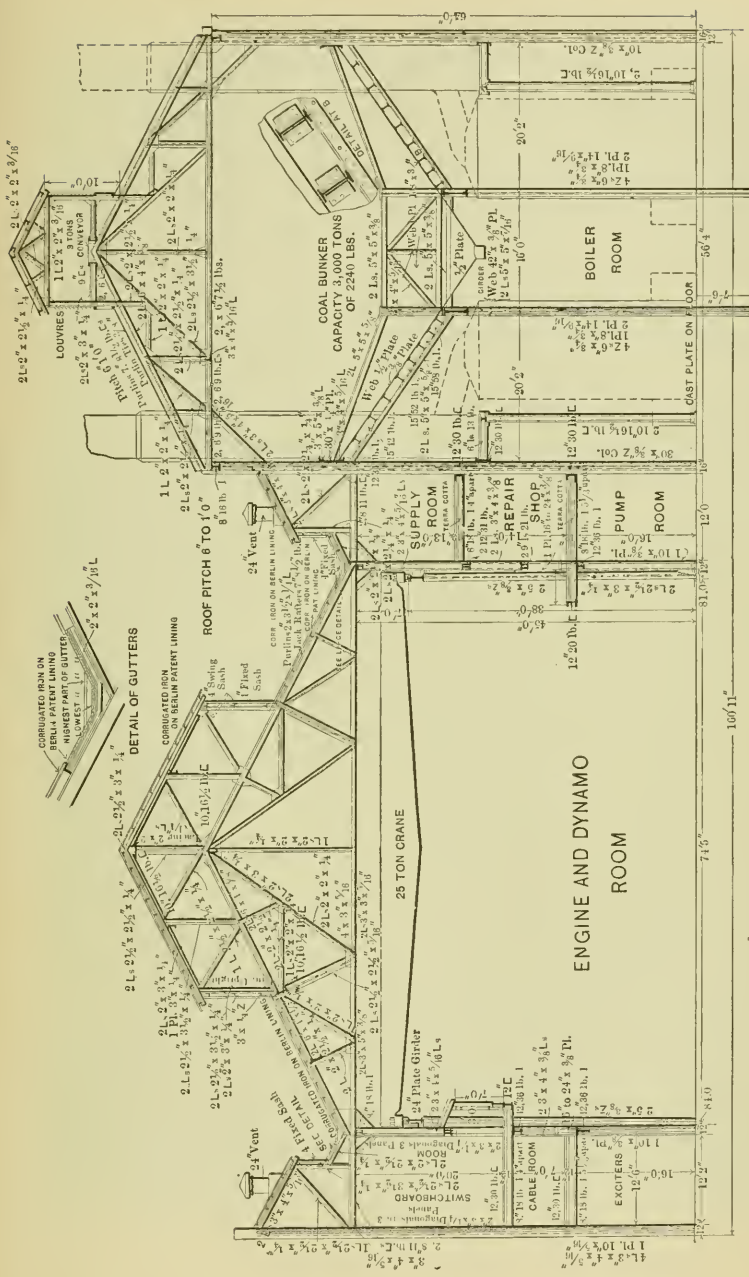
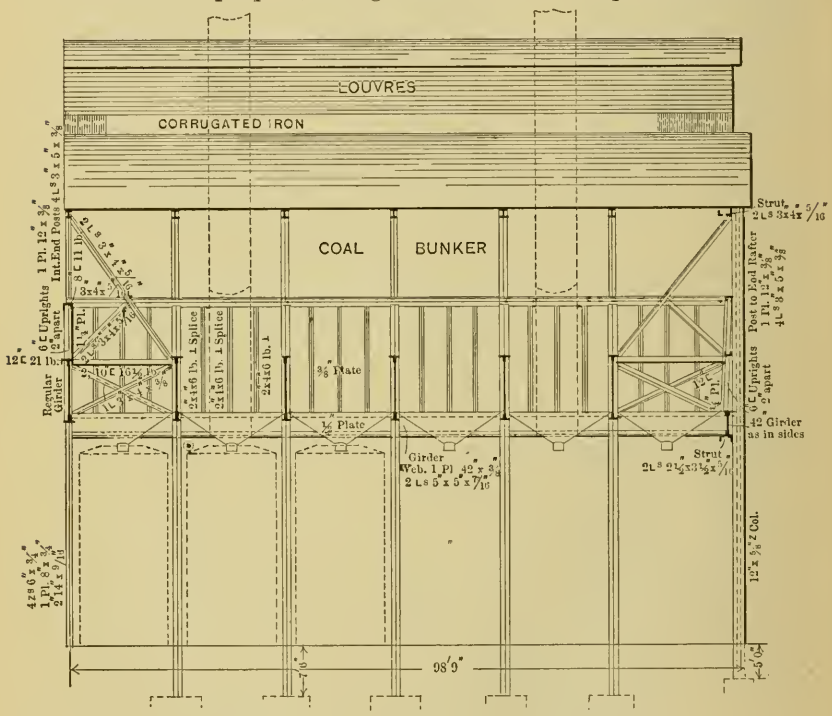

BOILER ROOM
COAL BUNKER CAPACITY 3,000 TONS OF 2240 LBS.
LOUVRES
CAST PLATE ON FLOOR
SUPPLY ROOM
REPAIR SHOP
PUMP ROOM
ROOF PITCH 6" TO 1'0"
DETAIL OF GUTTERS
CORRUGATED IRON ON BERLIN PATENT LINING
25 TON CRANE
ENGINE AND DYNAMO ROOM
CABLE ROOM
EXCITERS
SWITCHBOARD ROOM

FIG. 1.

It was the original intention to excavate down to the rock, which was only a few feet below the surface over the greater part of the area, remove the loose top rock and shale, and build up the foundations as shallow as possible. In the case of the engines it was the intention to build only enough foundation to carry them, and to hold them down by foundation bolts, which were to be secured by rusting in holes drilled in the rock. The rock found, however, was practically useless for the purpose, being full of mica and quite soft, as is



LONGITUDINAL SECTION THROUGH BOILER ROOM
FIG. 2.

most of the rock on Manhattan Island. It became necessary, therefore, to take out enough of this material to allow masonry foundations to be built of sufficient weight to hold the machinery without anchorage. The foundations for the four main engines on each side were built together as one foundation, and are shown in Fig. 6. They are of hard brick, laid in mortar, composed of three parts sand and one of imported Portland cement. Fig. 7 shows a single engine foundation in detail. Fig. 8 is a plan of the foundation walls and piers of

PLATE X.
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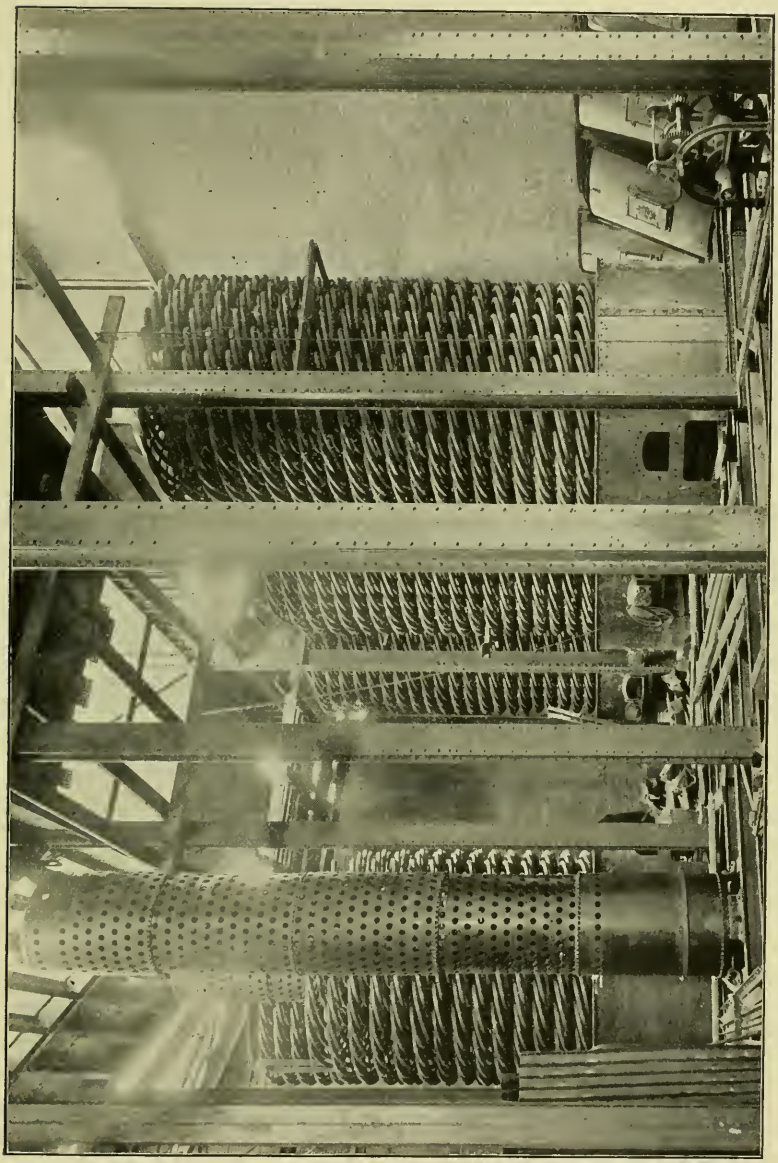
YORK ON ELECTRIC CENTRAL STATION.



the building, which were constructed in the same manner as the engine foundations.

The masonry was all done by day's work, while the steel frame was let out by contract, as were the piping and all other work of any magnitude.

The boilers are of the upright water-tube type and are in 600 H. P. units. Aside from being economical in the generation of steam, they were selected especially because they occupy so little ground room per unit of capacity. Six are now installed, and six more are required to complete the half of the station now erected. Plate X shows the arrangement of the boilers, as well as their appearance during erection.

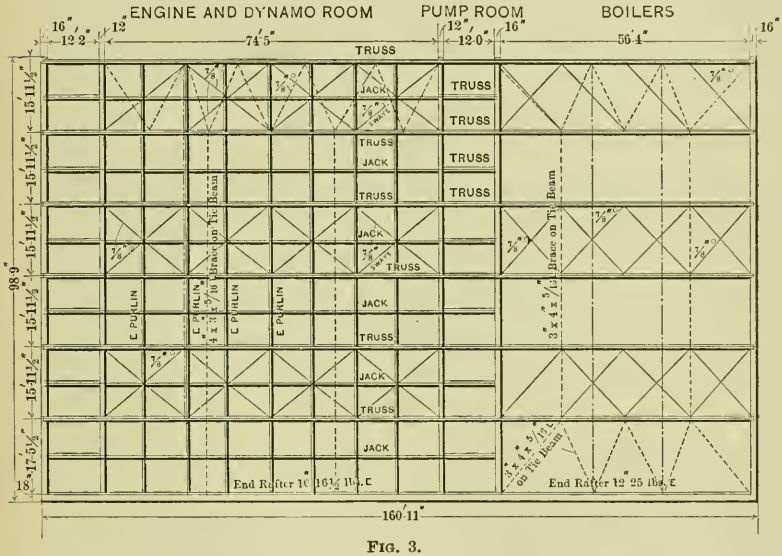


FIG. 3.

It will be seen that part of the boilers have all tubes in place, while the shell of one has just been set up, no tubes having been put in it.

The engine adopted is a Westinghouse double-acting, known as the "Columbian steepled compound," and is of the same type as those used in the lighting station at the World's Fair at Chicago. In fact, three of the engines at present installed were in service in that station. Fig. 9 shows one of these engines in section. The low-pressure cylinder is placed over the high pressure, and both pistons are connected to the same rod. The crank is enclosed in the same manner as is customary in the more familiar types of Westinghouse engines.

The low-pressure valve is operated by a fixed eccentric placed inside the crank case, while the high-pressure valve receives its motion from a shifting eccentric outside the crank case, operated automatically by the governor, which is placed on the shaft outside of the eccentric. The low-pressure valve is of the slide-valve type, while that for the high-pressure cylinder is a hollow piston valve, being constructed in this manner to allow the exhaust from the lower end of the high-pressure cylinder to pass up through it. On account of this construction it is impossible to cushion the valve itself, and this cushioning is accomplished by a plunger, receiving the same motion as the valve, which works in what is termed an inertia cylinder placed outside the crank case, as shown in the cut. This inertia balance is also used for working the engine by hand, the eccentric being first disconnected by throwing over a small hand wheel, and steam being admitted above or below the plunger by means of a small slide valve. The dimensions of these engines are as follows: Diameter high-pressure cylinder, 21½ ins.; diameter low-pressure cylinder, 37 ins.; stroke, 22 ins. The speed is 200 revolutions per minute and the rated horse-power 1 200 when operating condensing, with 150 lbs. initial steam pressure. Plate XI shows the four engines and dynamos already installed on the east side of the engine-room.

Each main engine is directly connected to a 600-kilowatt Westinghouse alternator by a rigid coupling, both engine and generator being set on a firm cast-iron bedplate, as shown in Fig. 7. The generator has but one bearing, the armature being swung between the engine and this single support. Four of these outfits are now erected in place, and four more will be required in the first half of the station. These generators are similar to those used in the lighting station at the World's Fair at Chicago, except that the armatures are wound for a different voltage and are slotted instead of toothed.

By this means the noise, which would be objectionable in a thickly populated district, is almost entirely done away with. The alternators are all arranged to give either single or two-phase current. This is accomplished in two of the four machines already in place by two separate armatures on the same shaft, set at the proper angle in relation to each other to give the form of current required. In the remaining two alternators the same result is accomplished with but a single armature.

PLATE XI.
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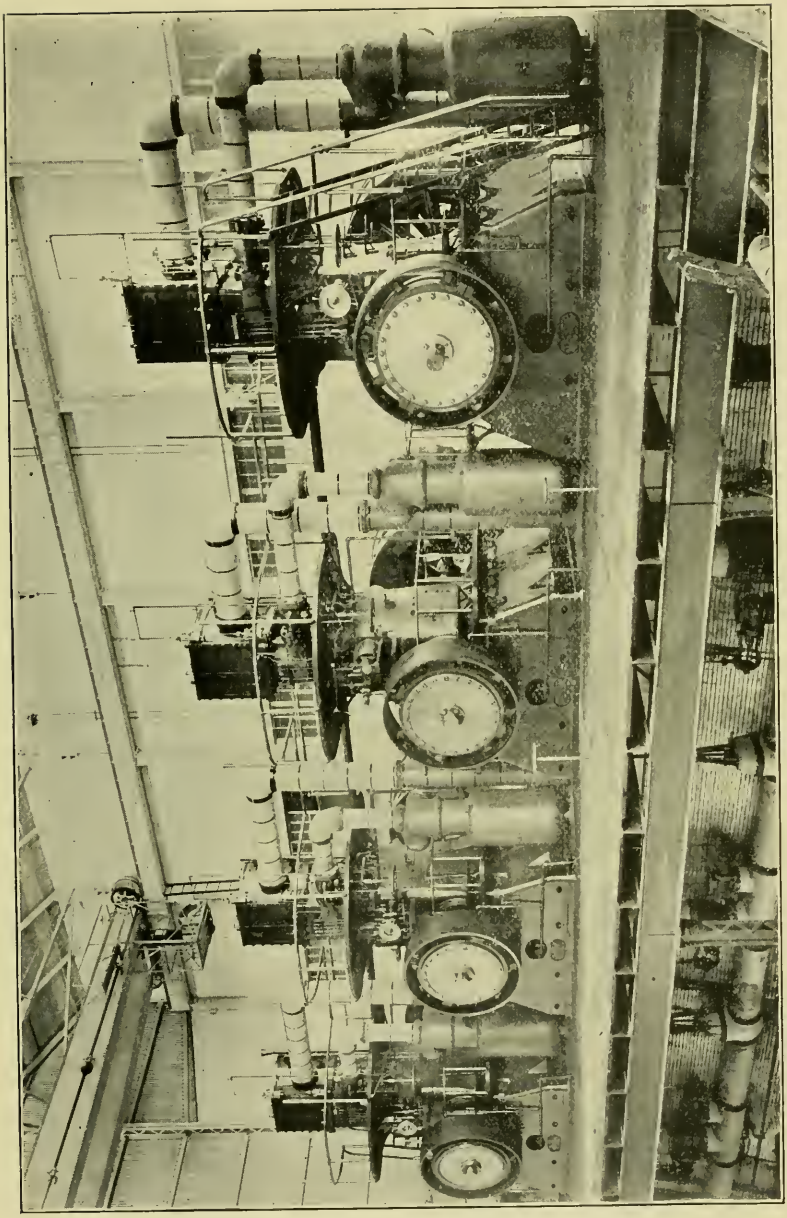

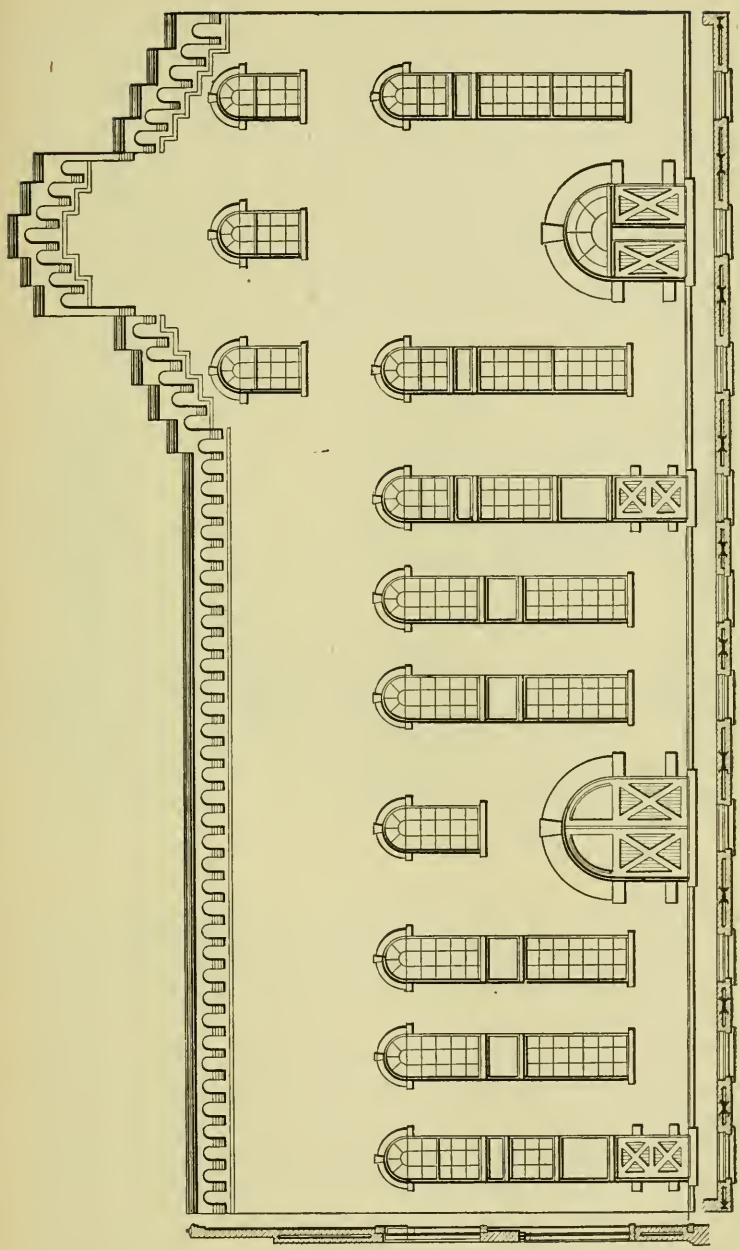

FIG. 4.

For exciting the fields of the alternators 75-kilowatt direct-current Westinghouse dynamos of the railway generator type are used. These are directly connected to 100 H.-P. Westinghouse single-acting tandem compound engines by means of a flexible coupling. Both the engine and the generator are too well known to require any description. Three of these outfits are provided for the eight alternators in the Twenty-eighth Street side of the station, and as each has sufficient capacity to excite four alternators, there is always one spare.

The feed pumps are arranged in the same manner as the exciters; that is, three are provided, any two of which are able to supply all the boilers in the Twenty-eighth Street side of the station at the time of

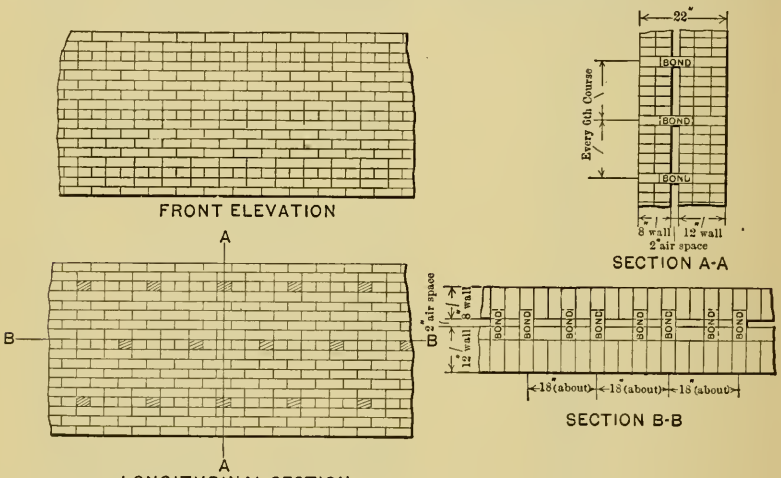

FIG. 5.

the heaviest load, leaving one spare. These are compound duplex pumps 9 and 14 x 8 x 10 ins. The steam supply is controlled by a regulating valve operated by the pressure in the feed mains, so that this pressure is always kept constant.

The entire engine and dynamo-room is spanned by a three-motor electric crane of 25 tons capacity, which is shown in outline in Fig. 1. The span from center to center of the rails is 72 ft. 3¾ ins., and the crane itself weighs approximately 45 tons. While a crane of this size and character in a power station may appear to some to be in the nature of an extravagance, the author considers it quite the reverse, and his opinion is borne out in this instance, at least, by the facts, as it has

saved in time and labor more than its cost in setting up the machinery now installed. Moreover, the first cost of the crane is very largely offset by the fact that its presence enabled the roof of the building to be made very light, just heavy enough, in fact, to provide for the wind and snow load, while it would have otherwise been necessary to make every roof truss strong enough to suspend the heaviest part of the engines or dynamos from it, thus materially increasing the cost.

The apparatus for handling coal and ashes is shown in Fig. 10. The

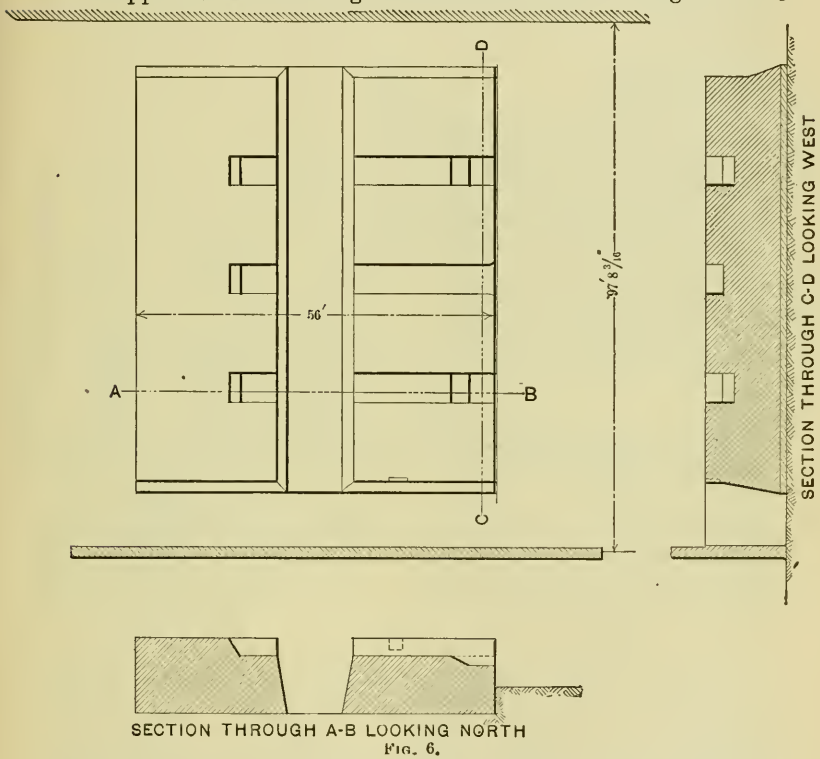

FIG. 6.

conveyor consists of an endless chain of gravity buckets, which are loaded by means of a filler and can be dumped at any desired point. The driver is in the north end of the ventilator over the coal bunker. The coal filler is in a vault under the sidewalk and the coal is dumped into this apparatus through a grating situated at about the street level. After being deposited in the buckets, the coal is carried up into the ventilator over the coal bunker and dumped into any portion of the bunker desired. From the hoppers in the bottom of the bunker,

the coal is spouted to the different boilers. The arrangement is such that the coal trims itself and will continue running down the spouts, as required and without assistance, so long as any remains in the bunker.

When the Twenty-ninth Street side of the station is built, a second conveyor running at right angles to the present one will be put in to bring the coal from the water front. As the company does not own the bulkhead on the Twenty-eighth Street side, however, it is necessary to cart the coal at the present time.

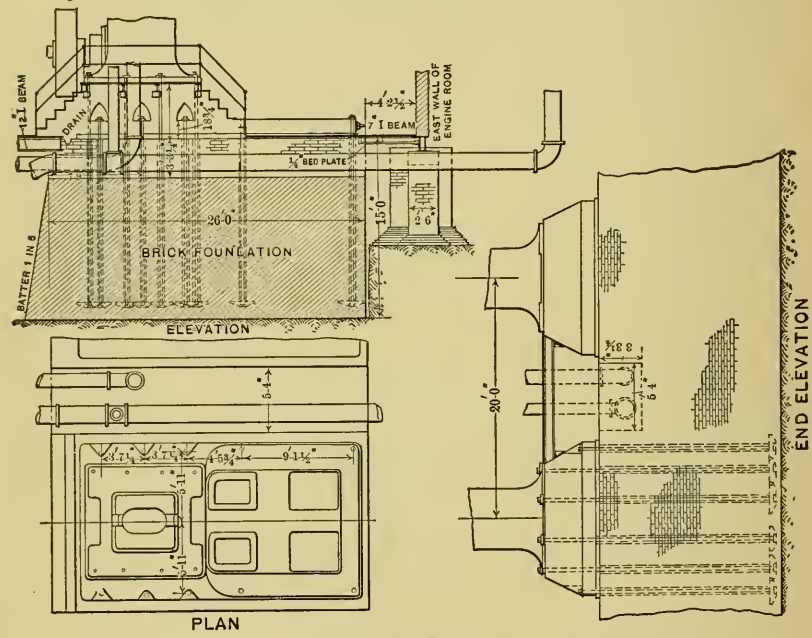

Fig. 7.

Under each boiler is an ash hopper delivering the ashes to a second movable filler, which deposits them in the buckets of the conveyor, when it is not used for coal. The conveyor dumps the ashes at a point from which they are spouted over to a tank in the southeast corner of the coal bunker. From this tank they are spouted down and out through the front of the building into carts. By this arrangement the same conveyor handles both the coal and the ashes, and neither is touched by a shovel, except that for the present the coal is fired by hand. This is a temporary arrangement, however, as the boilers are laid out and set with a view to the use of mechanical stokers, which will probably be applied in the near future.

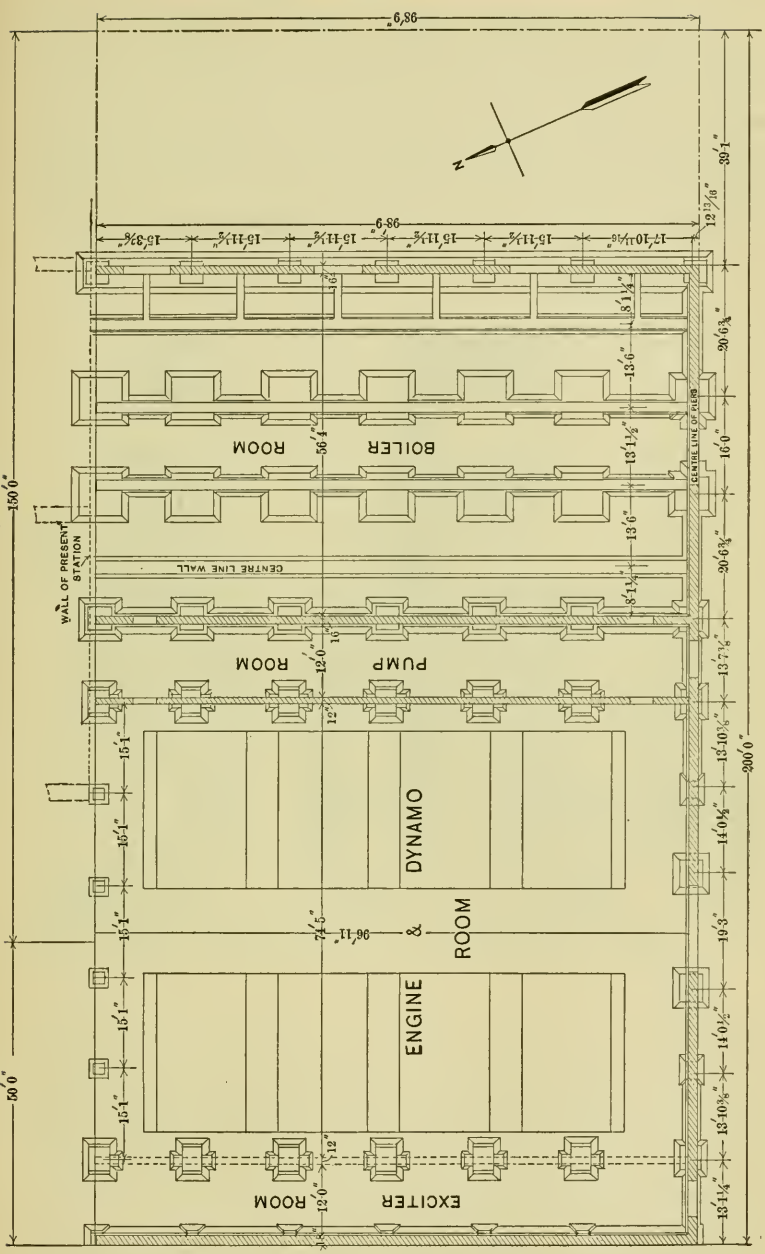
BOILER ROOM
PUMP ROOM
ENGINE & DYNAMO ROOM
EXCITER ROOM
CENTRE LINE WALL
CENTRE LINE OF PIERS
WALL OF PRESENT STATION
N
150'0"
50'0"
200'0"
98'9"
96'11"
74'5"
56'4"
39'1"
20'6¾"
16'0"
13'7⅞"
13'10⅜"
14'0½"
19'3"
13'1¼"

FIG. 8.

One feature of this station which is considered of special interest is the arrangement of the piping, which is laid out on what, for want of a better name, the author has termed the loop system. By this arrangement the size of the pipes required is reduced to a minimum, and without going to the expense of providing a duplicate set of pipes, steam can be shut off of any portion of the system without interfering with more than one engine or its equivalent boiler capacity. As this

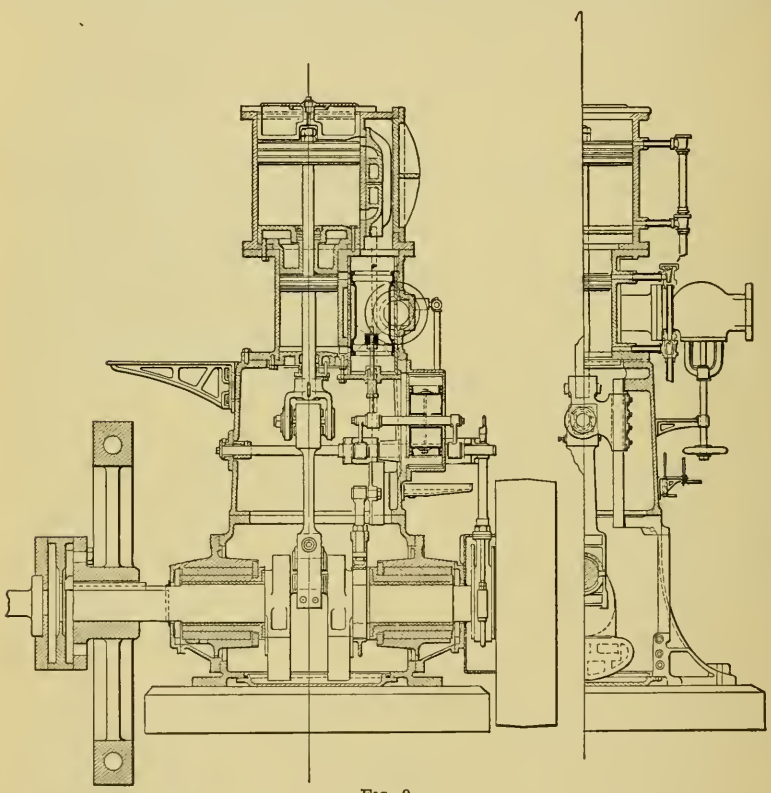

FIG. 9.

amount of spare is always carried, it follows that any portion of the steam-piping system can be cut out without interfering with the operation of the station, and the object of a duplicate set of pipes is thus accomplished without the expense of duplicate pipes and without occupying the space required for them.

The main steam and exhaust piping is shown in plan on Fig. 11 and in elevation on Fig. 12. A 16-in. header is run the length of the boiler-room

and a similar header is run the length of the engine-room between the two rows of foundations, and parallel to the boiler-room header. These are the largest live-steam pipes used and there will be none larger, even when the Twenty-ninth Street side of the station is complete and the entire 20 000 H. P. is installed. Each of these headers is divided into five sections by means of four gate-valves, located as shown, and each section of the boiler-room header is connected to the

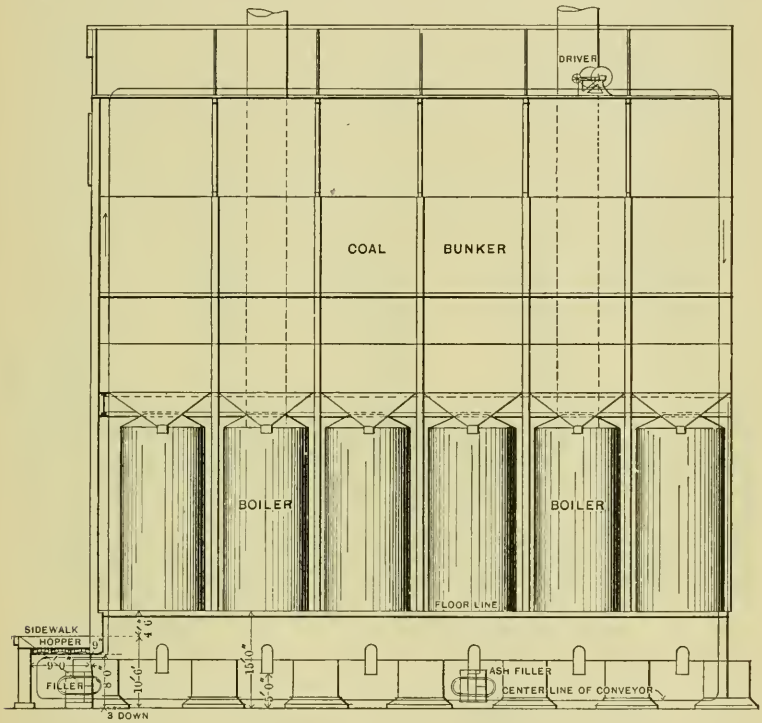

FIG. 10.

corresponding section of the engine-room header by a 14-in. branch rising from the top of one and discharging into the top of the other, a valve being placed on each end where a connection is made to the header. Each boiler has an independent connection to the boiler-room header, supplied with two stop valves, one in the customary position just beyond the safety valves, and the other at the point where the pipe enters the header. This second valve has its stem extended through the wall into the repair shop, so that in case of trouble any

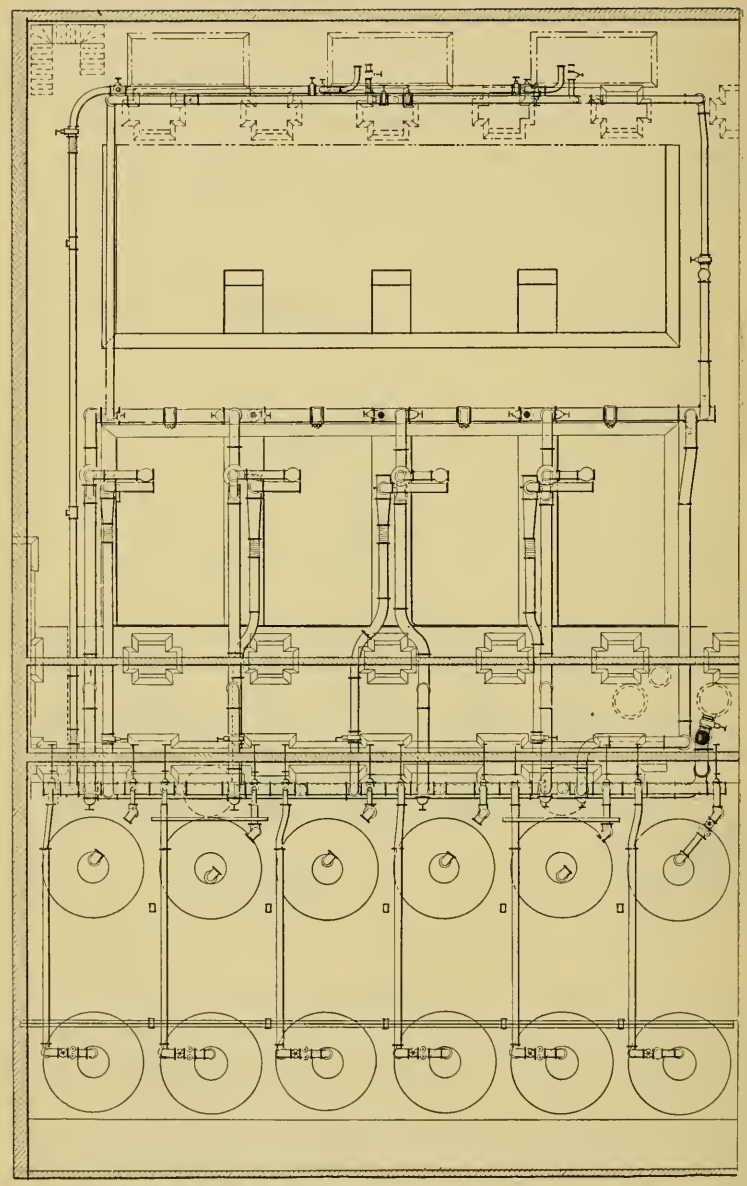

FIG. 11.

boiler may be cut out from a room having no communication with the boiler house.

Each engine on the east side of the engine-room is connected to one of the 14-in. branches previously mentioned, while outlets are left on the engine-room header for connections to the west row of engines as soon as they are placed in position. In case any section of the engine-room header is cut out, one engine connected to this section can be fed directly from the 14-in. branch, leaving only one which cannot be run, and in case any section of the boiler-room header is cut out, no engine need be shut down, as the one connected to the 14-in. branch can be fed back from the engine-room header. One 14-in. branch may at any time be cut out without interfering with the supply of steam, four of these branches being sufficient to furnish ample steam for all requirements.

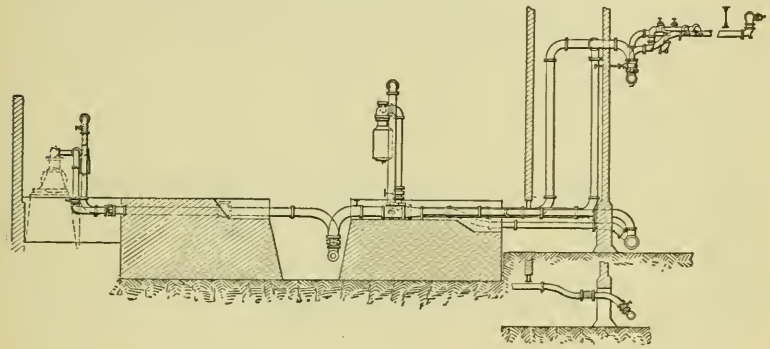

Fig. 12.

The steam pipe for the exciter-engines runs from the north section of the engine-room header around through the exciter-room, and back to the south section of the same header, as shown in Fig. 11. This forms another complete loop, which may be fed from either end in case of trouble with the other, or from both ends in case it becomes necessary to cut out its center.

The steam pipe for the pump-room is arranged in practically the same manner, as shown in Fig. 13, except that in this case the loop is fed not only at the two ends but also at three intermediate points, the same as the engine-room loop.

Another special feature of the steam piping of this station is the use of a combined separator and receiver, shown in detail in Fig. 14, as close to each engine as it is possible to place it. This receiver con-

tains six times the volume of the high-pressure cylinder of the engine, and as the latest cut-off is one-third stroke, it follows that at full load there is at the engine eighteen times the volume of steam required for one stroke. This reservoir prevents the excessive drop in pressure in the pipe when the valve of the engine opens and a corresponding rise above boiler pressure when the valve closes, and keeps a practically uniform flow of steam in the pipe in one direction, allowing the use of smaller pipes than would otherwise be required, and doing away almost entirely with the vibration in the pipes caused by the intermittent flow of steam.

Two low points are provided in the steam piping, from which to take any condensation that may be deposited. One of these is the

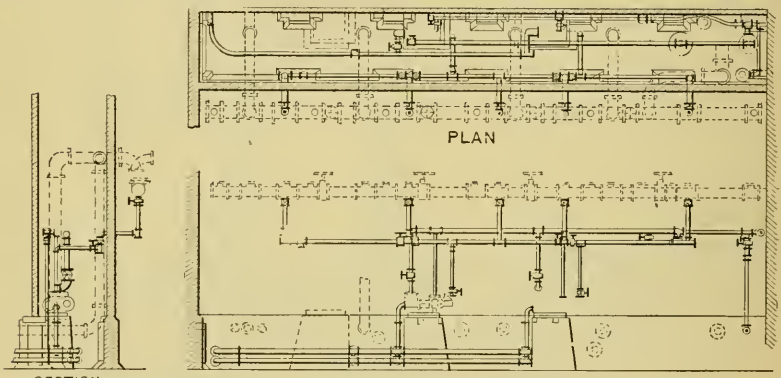

FIG. 13.

pump-room header, and the other a drip main under the engine-room header and connected to each of its sections.

Steam loops from both these points take all condensation and return the water to the boilers. A steam loop is also employed to return the water to the boilers from each of the separators.

Expansion of all live-steam pipes is provided for by long bends and swinging elbows, no expansion joints being used. The boiler-room header is supported by long hangers from the under side of the coal bunker, and is anchored in the middle of its length. The engine-room header is supported in cradles resting on rollers carried on piers, and is free to move horizontally in any direction. The vertical portion of each 14-in. branch connection is provided with a bracket ell at the bottom, which is supported from a pier by means of four 1½-in. set

screws, so adjusted as to take the weight of the pipe when hot. Hangers are also provided at the upper end of each of these vertical branches, but these carry no strain as long as the pipe is hot. When

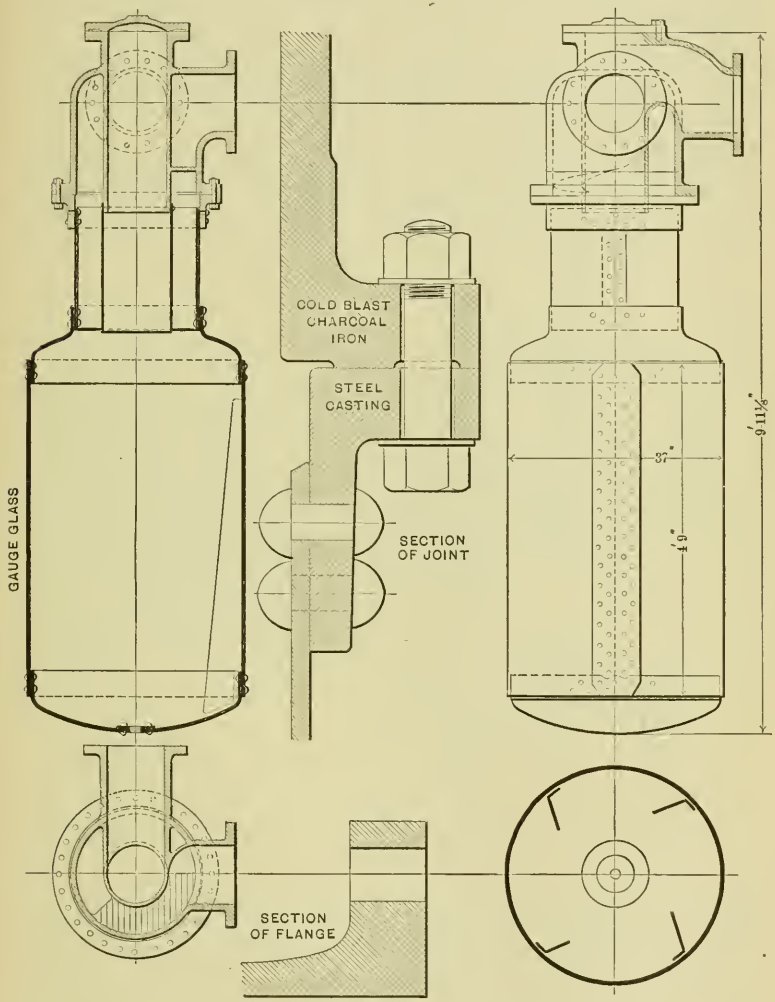

FIG. 14.

steam is shut off and the pipe contracts, the weight is transferred to the hangers, the bottom of the pipe lifting itself clear of the pier. This arrangement is for the purpose of confining the movement to the lower end of the pipe, where provision is made to take care of it.

All pipes, valves and fittings carrying live steam are made extra heavy to provide for 150 lbs. steam pressure, and were tested and made tight under 300 lbs. cold-water pressure after being erected in place.

The exhaust piping contains no special features and is clearly shown in Figs. 11 and 12. A second header will be provided for the west row of engines similar to that for the row already in place, and a third header will be provided, running directly to the condenser and connected to all the engines on both sides. Any engine may then be run either condensing or non-condensing. Fig. 15 shows the de-

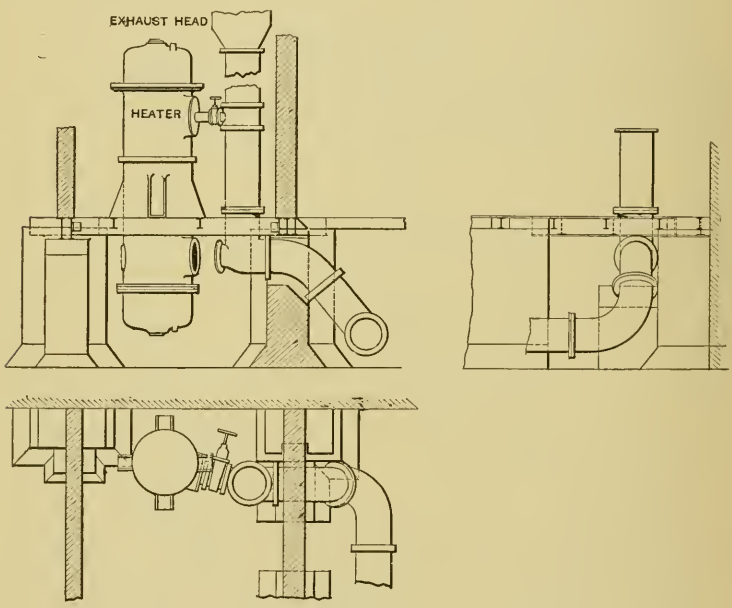

FIG. 15.

tails of the connection of the free exhaust to heater No. 2, and the details of the floor beam system in the north end of the pump room.

Fig. 16 shows the feed and blow-off lines. On the feed lines it will be seen that the loop system has again been employed. The mains make a complete closed loop in the center of the boiler-room, from which the boilers are fed on either side. This loop is fed at either end, the water to reach one end having to pass through the heaters and going directly to the other end. An inspection of the illustration

will show how any section of this system may be cut out without shutting down more than one boiler.

Although provision is made in this station for the switchboard and all the accompanying apparatus, the switchboard room of the old Twenty-ninth Street station has thus far been used, as it is advisable to have both boards in close communication while both stations are running and circuits frequently transferred from one station to the other.

The lead wires from the machines consist of lead-armored cables run in iron ducts directly from each dynamo to its panel on the dynamo board. On this board each alternator as well as each exciter has its own panel equipped with the customary appliances, including

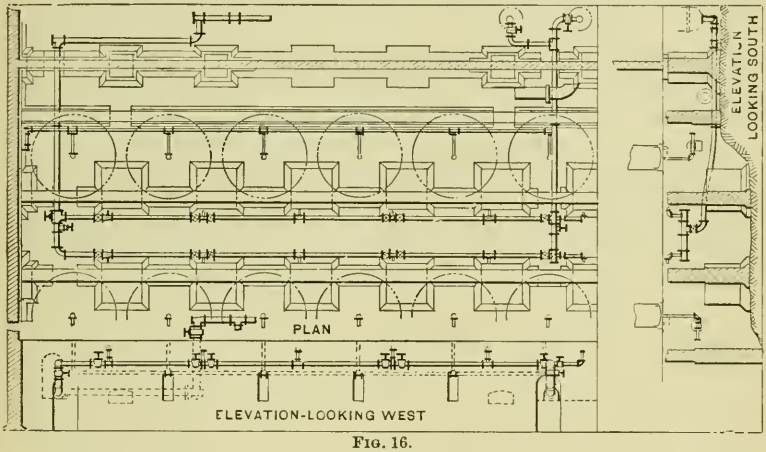

Fig. 16.

volt meter, ammeter, rheostat, fuse blocks, switches, etc. From each panel on this board connection is made to a separate pair of buss bars on the back and running the length of the feeder board, and here each circuit has a separate panel equipped with a volt meter, ammeter, continuous registering meters, etc. At a convenient height on each panel is situated a double-pole dynamo change-switch with the circuit leading out from the center and two flexible cables connected to each end. These cables can be plugged in on any pair of buss bars and a circuit transferred from one dynamo to another by a single throw of the switch. From the center of these switches the circuits run to a third board called the cable terminal board, where, after passing through fuse blocks and a second set of switches used for transferring any

circuit from its own panel to a spare panel provided in case it is desirable at any time to cut any feeder panel dead, connection is made to the ends of the cables, which extend down through the floor and out to the subways. The entire distribution system is underground, and occupies about 150 miles of ducts. The voltage is reduced to any desired pressure by means of converters placed in convenient locations, usually in vaults under the sidewalk. Continuous registering meters are employed to determine the amount of current consumed; they are of the Shallenberger type.

This station has been in constant operation since September 23d, 1895, when it was first started up, and the load on it has been continually growing heavier, as circuit after circuit has been cut over from the old system to the new. So far everything has operated in an entirely satisfactory manner, and none of the difficulties and annoyances usually incident to the starting up of a large plant have been experienced.

The author submits this paper with the belief that in this station are embodied a number of features somewhat unusual in central station design, and he will be pleased to discuss in more detail any special points which may be of interest to the members of the Society.

DISCUSSION.

Mr. Emery.

CHARLES E. EMERY, M. Am. Soc. C. E.—The details of the plant described in the paper show close study. While the larger work of the New York Steam Company was available, so far as the boiler plant is concerned, none of its general features were copied. There are details of steam engineering practice which are common property, but were perhaps never assembled until the steam company's plant was built. The author has combined these details in new and very satisfactory forms, and has employed the precautions which are necessary in case they are used, so that the paper will bear study as to the proper methods of overcoming difficulties in the construction of a plant of this kind.

The station of the New York Steam Company was probably the first in which the boilers were placed in tiers, one above the other. This has become the established practice for stations near the central part of a city where land is very expensive, although not to the extent of placing the boilers in four tiers as in the steam company's plant. Water-tube boilers of the Babcock and Wilcox type were installed, and the heating surfaces were undoubtedly as close together as they could be arranged by any plan, although it does not follow that other boilers are not available for central station work. The heating surface of the Morrin boiler used in the plant referred to in the paper is extended vertically to give the power, and a simple smoke pipe can be put on each boiler or a number combined, as in the Twenty-eighth Street station. The steam company's station was built originally to furnish 16 000 H. P. from the entire plant, but that capacity was actually obtained with the boilers on three floors and a few on the fourth, because they were run above their rated capacity. These figures refer to 16 000 H. P. with 30 lbs. of water, while the rating of 30 000 H. P. on the steamship *Lucania* is on the basis of about 15 lbs. of water, so that, on the latter rating, the steam station has a capacity of about 32 000 H. P. and will have more when the fourth tier is filled out. It is therefore the largest single installation of boilers ever made.

The system of steam pipes adopted by the author is practically that used in the distribution of water and gas in cities. It is unnecessary to make the pipe in each street large enough to feed the whole adjoining district. By sending the supply through several pipes, it is possible to feed around a break at any point just as is practiced in the method of electric distribution of the Edison system. The method has been well worked out by the author, and enables any single boiler or engine to be cut out while keeping the pipes small. The loop system of feed pipes is perhaps not so novel. In the construction of the steam

Mr. Emery. company's station, it was considered unnecessary to have the complete duplication of pipes which is now generally adopted, but what is called a donkey system in marine engineering was installed instead. A 6-in. connection was made at the rear of each 250 H. P. boiler, which furnished steam for the pumps and other accessories. By increasing the boiler pressure, steam could be taken through the back pipes and fed to the street mains, as the boilers were of ample strength to carry the additional pressure. This was done on several occasions.

In the electric light stations designed by the engineers of the Edison Electric Illuminating Company of New York City, the speaker recommended for the engines a foundation isolated from the building. This plan had been followed by the president of the company in his house, where the foundation was placed on a bed of sand and did not produce vibrations in the building or in the adjoining houses. Owing to the success of this small plant it was recommended in the uptown stations, with certain features adapted to the locations. The rock was excavated to the necessary depth and covered with a few inches of sand supporting a mass of masonry and concrete to which the engines were bolted. This isolated foundation was nearly the full size of the building, but separated completely from the walls and columns. The aim was to avoid noise more than jarring, and the results were satisfactory, as has also been the case with the author's method of anchoring his engines to the bed rock.

The saving of room in the Twenty-eighth Street station is due in part to the increase in the size of units. In the design of dynamos and electric generators there is great economy due to making them in large units. The generators in this station were regarded with doubt by some engineers before they were tested at the Columbian Exposition by the speaker, as chairman of a committee of the judges in the Department of Electricity, but the test was highly satisfactory, and the engines, notwithstanding their large size, ran regularly at the remarkable speed of 200 revolutions per minute. The steeple form of engine will operate on less floor space than a double engine, but the latter has advantages which will probably cause it to be used more than the single steeple form.

Another improvement adopted in the plant has a marked influence on the economy of electric distribution. It is known that the output through a certain set of conductors is proportioned to the square of the voltage. In this station the generators were wound originally for 2 000 volts, but they were rewound after the close of the Columbian Exposition, and the increased voltage enables the current to be distributed over the entire city without much loss of pressure. The generators in the old station are wound for only 1 000 volts, and give less than one-fourth the amount of electric energy over the same conductors as the new plant. This increase in capacity, due to higher voltage,

may be sufficient in itself to earn dividends, where. perhaps, they could not be obtained under the old conditions. Mr. Emery.

The method of handling coal and ashes in the author's plant, when compared with that in the station of the New York Steam Company, shows the marked improvement in such practice which has been made in recent years. When the latter station was built, the coal conveyor used so successfully in many recent electric light plants had not been brought out. In the steam station coal is dumped into cars at the sidewalk level, the cars are run back, raised by an elevator, run over the coal bins and dumped. The ashes are collected in the basement in similar cars, run to the rear, raised on the elevator and dumped down an ash chute. The system of conveyors in use at the Twenty-eighth Street station is an improvement over this plan, as it is operated with fewer employees.

George R. Hardy, M. Am. Soc. C. E.—The modern character of the plant described in the paper, as respects boilers, piping, separators, engines, generators and the storage and handling of fuel, and the assured success of such a compact installation on a plot of ground containing 32 000 sq. ft., or 1.6 sq. ft. per horse power, command attention. As a unit for comparison, the engine and its generator may be taken. The 32 000 sq. ft. to be occupied by sixteen such units provides 2 000 sq. ft. for each, inclusive of the proper proportion of space occupied by boilers, exciters and other accessory apparatus. The plan shows that the coal storage area over the boilers measures about 12 000 sq. ft. for a total capacity of 6 000 tons, which is equivalent to a load of half a ton per square foot. This area is usually exceeded in railway coal stations, although in some of the larger stations of 40 000 or 50 000 tons capacity it is about the same. The total horse power represents, after allowing for exciters, 1 200 H. P. for each unit, and is delivered to the generator without any loss of power incident to the use of belting. A comparison of this station with the old plant on Twenty-ninth Street shows that a unit in the new station occupies only 31% as much area as in the old. The speaker doubted if more compactness per horse power could be shown outside of marine engines. Mr. Hardy.

With respect to railways this plant offers suggestive comparisons. Railways are frequently compared with reference to length, but their ratio of equipment is also a matter of much importance. Locomotives demand the first attention. Although they vary widely in efficiency, few roads of large extent are to-day without advanced types of engines combining points of dimensions and usefulness closely approximating the units of the author's station. The newer engines, being more complete, are put in the hardest service, while the older ones are distributed over the lines for the lighter service better fitting their capacity. When the engines are housed the land required measures

Mr. Hardy. about the same for engines as the land per unit at the Twenty-eighth Street Station, not allowing any space for bridges or other structures. An engine-house recently designed, which contains 34 stalls, would require the purchase of a square lot containing 70 756 sq. ft., 2 080 sq. ft. per stall, or about the same area per unit as in the author's central station.

The accompanying table gives a statement of leading data relative to the Columbian steeple compound engine, and three locomotives, viz., the Schenectady consolidation, the New Haven consolidation and the New York Central No. 999 type:

	Columbian.	Schenectady.	New York, New Haven and Hartford.	New York Central.
Weight, pounds.	200 000	189 100	234 850	204 000
Cylinder, diameter, inches....	21½, 37	21	21	19
" stroke, inches.....	22	26	26	24
Steam pressure, pounds.....	150	170	180	190

The weights are for locomotives and tenders. The heating surface provided for each Columbian engine is 9 000 sq. ft., and that of the Schenectady consolidation locomotive is 2 124 sq. ft.

Railways command such different facilities for securing fuel that no constant supply can be assumed, and even on a single line the provisions vary with circumstances. Some lines near collieries, or located so as to obtain coal conveniently, will carry less than a week's supply, while other lines provide storage capacity for several months' consumption. The coal bins on one road are often furnished with a supply for a year. The electric light station described in the paper, which has water facilities, ought to have a storage capacity for thirty days. This would indicate that the designer anticipated a consumption of 200 tons a day when running the station near its full capacity, thereby averaging 12½ tons per unit daily. If it is conceded that the economy of a plant of this type with compound engines is such that only half as much coal per horse power is used as is consumed and wasted by a locomotive, then each unit will furnish as much power as a locomotive using 25 tons of coal. Locomotives using three tons a day would compare with the station unit as eight to one, which makes the work done at the station under full load equivalent to the work of 128 locomotives.

Some recent tests of consolidation locomotives having 96 000 lbs. on the drivers, and weighing 185 500 lbs. with the tenders, show that 800 H. P. was the greatest amount developed. Where these locomotives worked on heavy adverse grades for 27 miles, with an average resistance of 10 250 lbs., these same tests show that the average horse

power in that distance did not exceed 400, and the down grades on the remainder of the run would probably reduce the work done 50%, making it 200 H. P. The conditions are approximately paralleled in the electric light station where a maximum output, doing most of its work in seven hours, nevertheless averages for 24 hours about 25% of the highest output. Mr. Hardy.

Tests with a 17 x 24-in. passenger locomotive hauling a train weighing about 224 000 lbs. gave the following results:

Speed, miles per hour	50	55	60	65
Boiler pressure, pounds	110	135	125	139
Indicated horse power	229	190	260	350

An electric railway service of thirty 10-ton cars running with frequent stops requires about $16\frac{2}{3}$ H. P. per car. This would indicate apparently that about 670 H. P. is needed for a train of the usual size, but the service of the electric roads has such frequent stops and adverse grades that the same load can probably be moved over a steam road with an expenditure of not over two-thirds this amount.

By means of a diagram showing the position in respect to time of the 134 passenger and freight trains on a certain schedule, it can be shown that in no case are there over 22 trains, $16\frac{2}{3}$% of the total number, on the road at the same time. Assuming that each train requires 440 H. P., the total will be 11 000 H. P., about half the capacity of the electric plant described in the paper. As the allowance of 440 H. P. is somewhat in excess of the actual power as determined by tests, it would seem that half the units of the electric lighting station would furnish sufficient power to operate such a division of standard steam railway, and the station would accordingly have a duplicate set of units for emergencies. If these eight units were located one in every 10 miles they would provide for 128 trains daily on a division 80 miles long, and would consume 100 tons of coal, half the quantity mentioned previously.

E. E. Russell Tratman, M. Am. Soc. C. E.—The author refers to the importance of economizing in space occupied, where land is so valuable as it is in New York, so it would be of interest to know whether any consideration was given to the question of erecting a building for manufacturing or office purposes, which would be a remunerative investment, as well as for power purposes. The location at Twenty-eighth Street, near the East River, is probably not a suitable one for an office building, but may be adapted for certain industries, power for which could be furnished by the electric plant. The plans do not indicate any provision for an extension of this kind. The Metropolitan Traction Company of New York has erected a large office Mr. Tratman.

Mr. Tratman. building over its power plant at Houston Street and Broadway, and the speaker had not noticed any sound or vibration from the machinery in the basement. On the other hand, the Third Avenue Railroad Company in the same city has a large one-story power house on the Bowery, which is of somewhat massive appearance, and gives the impression that it is intended to form eventually the lower part of a tall building.

In regard to preventing the transmission of vibration due to the machinery, there have been cases where trouble has occurred from the vibration being transmitted through the foundations and the rock upon which they rest to some considerable distance. This has necessitated the introduction of some form of cushioning in the engine foundations by means of asphalt, timber, etc. In this case no such cushioning seems to have been considered necessary, owing, perhaps, to the easy-running qualities of the engine used.

The nest of closely packed loops of small tubes through which the water circulates in the boilers would seem to make it imperatively necessary to purify the feed water, concerning which the author is silent, since the cleaning and renewing of the tubes and the locating of any defective or leaky tube must be a matter of considerable difficulty. While boilers of this kind are in somewhat extensive use, the speaker had a certain prejudice in favor of a type giving greater accessibility for inspection, cleaning and repairs.

The increasing number of accidents from the bursting of steam main pipes and the serious results of many such accidents led the speaker to inquire what material is used for the steam piping and elbows in this plant, and what provision, apart from the receiver or steam chamber shown in Fig. 14, has been made to ensure against the water hammer which seems to have been considered as the most common cause of the failure of such pipes. The position of stop valves operated from outside the boiler room is a wise precaution, and it is encouraging to read of the careful way in which the piping system has been designed, as this is a detail in steam plants which is too often given but slight attention, and is too often defective in efficiency and safety.

CORRESPONDENCE.

H. W. York, Jun. Am. Soc. C. E.—The question of erecting a building, the upper floors of which could be used for manufacturing purposes, was taken into consideration at the time the station was designed, but there were several reasons which made it appear unadvisable to carry out this construction. On account of the height of the boilers and the space occupied by the coal bunker, the engine-room would be the only portion of the building over which it would be practicable to construct lofts. As this room is 74 ft. 5 ins. between walls, it would be very expensive to provide floors over this space strong enough for manufacturing purposes, unless the girders were supported by intermediate posts, which would interfere with the space in the engine-room and make it impossible to span the entire room with one crane. Again, on account of the advisability of keeping the front of the building closed tight, it was necessary to depend almost entirely for light and ventilation on what could be obtained from overhead, and if this were cut off it would have become necessary to provide mechanical ventilation and use artificial light at all times. Various other questions also entered into the consideration, tending to strengthen the conclusion that it would not be advisable to try to construct a building for anything more than the company's own purposes. Mr. York.

The question of cushioning the engines is one that was studied very carefully, and all the data on this subject which could be obtained were gone over thoroughly. The conclusion reached was that while cushioning between the sides of an engine foundation and the adjacent foundations of a building serves very well to prevent any lateral transmission of the vibration, attempting to cushion under a large engine is apt to increase vibration rather than diminish it, and is also liable to cause serious trouble by allowing the engine to get out of line if the material is not thoroughly confined. In case the danger of throwing the engine out of line is avoided, it then becomes a case of taking up not only the vibration due to the moving parts, but also that due to the entire engine vibrating on the cushion.

While cushioning in some instances may prove effective, the author is decidedly of the opinion that the better method, in most cases, is to provide a foundation heavy enough to take up all shock, and then fasten the engine to this as firmly as possible.

This opinion is certainly borne out by the results in the case in question.

There has been no attempt made in this station to purify the feed water, Croton water being used just as it comes from the mains. While it possibly requires a little more attention to keep a bent-tube

Mr. York. boiler clean than a straight-tube boiler, there is no difficulty, at least where Croton water is used, in keeping them in good shape without taking any extraordinary precautions, such as purifying the feed water. Although the tubes seem to be very closely intertwined they are all easily accessible, and any tube can be readily replaced in a couple of hours.

The steam pipe used in the plant is all full weight lap-welded iron pipe with the exception of the two 16-in. headers, which are built of steel boiler plate. The valves and fittings are all cast iron, but are made extra heavy to provide for the pressure carried.

The provision made to ensure against water hammer is the only provision that can be relied upon to accomplish the purpose, viz., to make it impossible for a quantity of water to collect anywhere in the system. This result is obtained in this station by having all pipes drain in the direction of the flow of steam to low points from which the water is removed as fast as it collects and returned to the boilers by means of steam loops. So thoroughly has this system been carried out that when steam was first put on the cold pipes there was not a sound, not even a snap or a crack anywhere. The author is aware of the fact that steam pipes are often considered to be extremely dangerous, tricky and liable to fill up with water and burst without any provocation, but it has been his experience that, if given half a chance, they are not only safe, but less troublesome than almost any other portion of the apparatus of a power station.

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TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 780.

CONCERNING FOUNDATIONS FOR HEAVY BUILDINGS IN NEW YORK CITY.

By CHARLES SOOYSMITH, M. Am. Soc. C. E.

PRESENTED APRIL 15TH, 1896.

WITH DISCUSSION.

As an introduction to what follows, it would seem well to describe briefly the chief conditions met under the surface in the parts of New York City where the high buildings recently constructed and proposed are situated. To this end a map of the city below Twenty-third Street is given in Fig. 1, which shows the principal topographical features of that territory before it was built upon. The dotted portions near the shore line show the made land, which it will be noticed is generally from one to two blocks in width. This map shows that Broad Street to above Exchange Place and Maiden Lane to Nassau Street occupy what were formerly inlets from the East River, and that there were ponds east and north of City Hall Square. There seems to have been a natural waterway practically all the way from the North River following about the line of Canal Street and into Collect Pond, and from thence to the East River near Chambers Street. Notice should be taken also of the location of Minetta Stream, which was formed

near the present junction of Eleventh Street and Sixth Avenue. This stream flowed through what is now Washington Square and into the Hudson a little below Houston Street. The lines indicating elevations must not be allowed to mislead; in many cases the comparatively elevated portions are land made by the processes of Nature in comparatively recent times, it being rather the exception in the lower part of the island to find other than a drifted or deposited material within the distance below the surface at which the foundations of buildings are usually built. This upper material is mud, silt, sand, usually from fine to very fine, and sometimes gravel. The hard stratum below it, where not rock, is generally an exceedingly firm and compact surface of silt, clay and gravel containing stones of various sizes, which is generally called hard-pan. This is so hard that, in reports on the borings taken to ascertain the nature of the underground material about the city, it has generally been referred to as rock. The so-called bed-rock in the profile prepared for the proposed rapid transit tunnel road is this material, in all probability, in lower Broadway. In its natural condition in place it is so hard that for all the weights that can come on it through the construction of buildings it may be considered absolutely unyielding. It cannot be dug up except by pick or chisel, and in large excavations can be best loosened by blasting. It may be well to state here that in the case of the Manhattan Life Insurance Building, the weight of which is borne on fifteen caissons proportioned to carry a pressure at their bases of 10.8 tons per square foot, some of them rest on solid rock and some on the compact stratum mentioned, and that not the slightest settlement can be discovered. As to its safe bearing capacity, there can be no question but that it is well in excess of what under the present building law may be put upon it by means of a concrete base, 150 lbs. per square inch or 10.8 tons per square foot.

As the thoughtful engineer views the building methods prevailing to-day, it would seem to him that there is perhaps no kind of construction where the force of habit has so disastrously hampered a proper and quick readjustment to new conditions. Buildings of the heights usual until recently seldom put upon the earth supporting them weights greater than 3 or 4 tons per square foot, and generally much less than this. The weights were distributed and spread without difficulty by the common methods that had always been used. Of

course there was occasionally some particular case sometimes treated in an exceptional manner, but these were never important or numerous enough to bring about any radical change, either in the general carelessness with which the matter of foundations of buildings was considered,

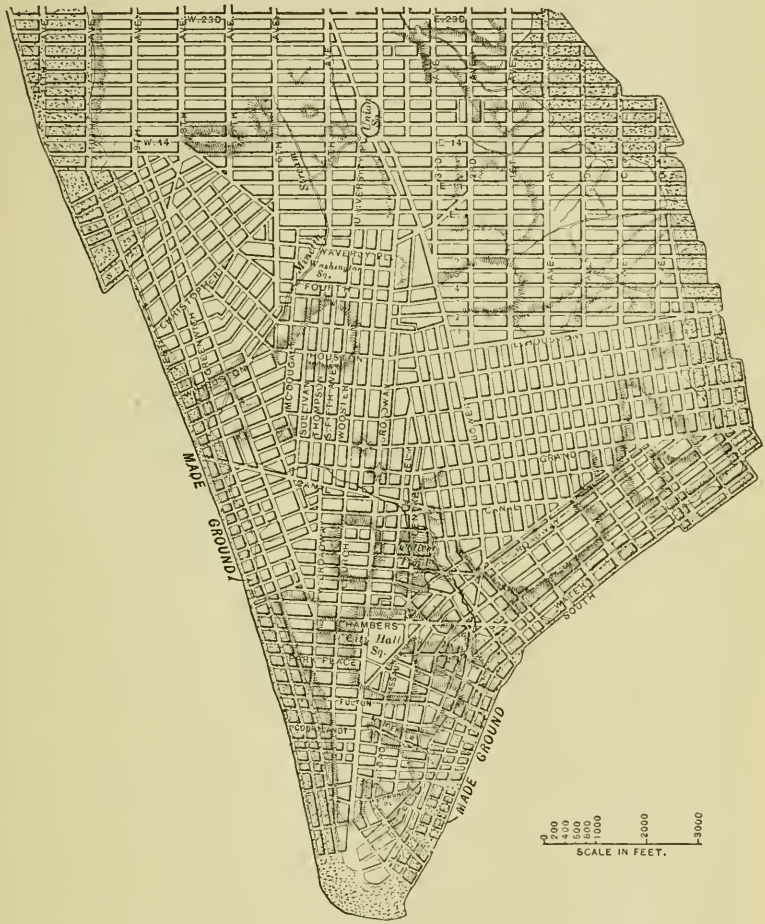

FIG. 1.

or rather not considered, or in the method of constructing them. Nor was this necessary, because the earth held reasonably well the buildings built upon it, and, too, the cost and probable life of the buildings did not warrant a greater expenditure upon the foundations. The

weight upon the soil did not compress it to such a degree as to cause serious settlement, nor did it put the material under a pressure which would press it outward or upward if given an opportunity by a near-by excavation. With the sudden doubling of the usual height of buildings, and the consequent great increase of weight, the old foundation methods became, in many cases, entirely inadequate. If pursued, serious settlement was invited from overloading, and the danger of letting down the underlying material by lateral flow became considerable. In fact, the problem of foundations for a heavy building, where they are not carried to rock or hard-pan, has now become (and this fact seems to be strangely overlooked) one very largely of what vent may be given for the underlying material by excavations in the vicinity. It is not sufficient that the weight put upon the soil should not be so great as to cause serious settlement. Under the conditions existing this is not generally difficult, but the material must not be put under a pressure that will cause it to squeeze out from below in case an opportunity is afforded by deep excavations nearby. It will become apparent ere long that the erection of heavy buildings resting upon sand or soil, which they unduly overload, without carrying their foundations on piles or to solid material, has in many cases worked a serious wrong to owners of the adjoining lots, besides being itself a short-sighted economy, because of trouble and lessened value of the buildings so built. For instance, the author knows of one building of great weight, a very excellent and costly structure with a carefully built foundation, which, however, rests simply upon the fine sand into which it has settled slightly. This settlement, as was anticipated, has been uniform and has caused no damage, but the owners are aware of the danger of serious or disastrous settlement that may take place from the operations of putting in foundations of buildings in the neighborhood, even though at a considerable distance from them, for at no great depth below their footings there is material, if it be like that revealed by the nearby operations, which would be likely to squeeze out under the existing pressure, if given a vent, especially if at the same time there is any exciting cause from jarring or pumping. The shoring and underpinning of such a building, while possible, would be vastly more difficult and costly than for a building of but a few stories' height. It would, for instance, cost probably more than $100 000 to care properly for the building in question when the property adjoin-

ing is built upon, unless the height of the new building were kept considerably below what would now be the height of the most suitable improvement of the property. Under the present building law, when a structure is built alongside another and this latter has its foundations not less than 10 ft. below the street surface, the responsibility and cost of caring for the old building must be borne by the owners of the new adjoining one. Thus in the case just cited, when the property alongside the building mentioned is adequately improved, the builders have to grapple with the problem of supporting and probably building a new foundation under this building. Thus the fact that this building, while conforming to the building laws, has put the soil beneath it under unusually great pressure, will cost the owners of the adjoining lots at least $100 000, beside any damage that they may cause the old building. The question very forcibly presents itself, whether in the absence of specific statutes on the subject, the common law cannot and should not be invoked to prevent any owner from putting the soil beneath a new building under such unusual pressure that it is under a dangerous tendency to squeeze laterally and thus materially limit his neighbor's future improvement, or else put him under great additional expense in carrying it out. In equity he should certainly be required so to use his own lot as not to injure that of his neighbor. It will be strange if this matter does not soon come up for consideration.

Where soils are overloaded there is extraordinary potency in slight vibration to cause settlement where the conditions are favorable. Hoisting engines are frequently found to be exciting causes in producing unequal settlement in the absence of unequal loading. In one case in lower New York where a very massive and well-built structure resting upon gravel and sand cracked after it had been in use a considerable time, the only new condition introduced to which it seemed possible to attribute the settlement manifested was pumping clear water from a nearby driven well. It seemed reasonable to assume that the water flowing through the sand made possible a slight readjustment of the particles which settled into closer union under the vibration of the machinery in the building.

A most interesting thing often shown by the settlement of buildings is the frequency of movement of the entire mass of soft material in the vicinity of rivers. There is a locality in New York City where there is

good reason to believe such a motion to be taking place. For instance, a certain large and well constructed building resting upon a well designed pile foundation, and where the piles were driven 40 or 50 ft. into the silt, started to settle, probably before it was completed; at any rate, it settled from almost nothing at the end farthest from the river to some 2 ft. at the end nearest the river, making a practical wreck of part of the structure. This building was some 200 or 300 ft. from the river. An examination of the bulkhead wall showed that the piers had apparently acted as buttresses to hold it back, but that between the piers it was bowed outward. This, in connection with other evidence, seemed to point indisputably to a movement of the entire mass of the adjacent material toward the river, occasioned by the weight of filled material and buildings, and it seemed probable that this movement was shared by the silt to a considerable depth below the filling.

The first method that naturally comes to mind for providing a better foundation than can be done by simply spreading the bearings on the earth at customary depths is that of driving piles, and where there is reasonable certainty that these will always remain wholly submerged, this is generally the best possible foundation, considering its cost, for buildings of considerable but not of the greatest weight. The New York building law permits a load of 20 tons per pile, and engineers will consider this, when fair-size piles are used, to be within good practice; but there are many cases where it is certainly a great mistake to take the aggregate bearing capacity of a pile foundation to be the sum of the safe loads on the individual piles composing it. The sustaining power of a single pile may be said to be practically the sum of the friction which holds it on the sides, and the resistance to penetration at its lower end. It is obvious that in very many cases where a large number of piles are driven close to one another, the piles simply replace so much material which has been displaced to make room for them. It is true that the material remaining about them may by the process of driving become so much more compact that it increases the friction and perhaps the resistance to penetration at the lower ends of the piles; but if the stratum below the piles be at all yielding it is probably true that the bearing capacity of the foundation is the bearing capacity of the stratum below the piles plus the friction of what might be considered the outer side surfaces of the entire mass penetrated by the piles. In other words, the piles merely

replace so much yielding material and transfer the load to the stratum beneath them.

A practical difficulty which is apt to hamper the construction of a good pile foundation for a building very seriously where there are a great number of piles close together is that, after driving the first few, the material near them becomes so compact that it is exceedingly difficult and sometimes impracticable to get the remaining piles down to anything like the contemplated depth. It may be comparatively easy to drive the first one, say 30 ft., and may become exceedingly difficult to drive those put down later to a depth of more than 10 or 15 ft. This difficulty is readily overcome where the use of the jet may be resorted to. This is often dangerous in proximity to other buildings, and, too, it sometimes seriously cracks the adjoining building to drive piles close to it.

With regard to the piles remaining permanently submerged, there have been some rather startling experiences in New York City. There is one building, the value of which is probably in the neighborhood of $1 000 000, built upon piles, the upper ends of which it was found before the completion of the building would be shortly exposed, and it is said that it was decided to enclose the entire foundation in a dam, so to speak, in which the water is kept by pumping at a safe height to cover the piles. One of the leading builders in New York a short time ago removed a building which rested on piles driven but a dozen years ago, and he found them seriously decayed. Throughout the gravel and sand deposits of the lower part of New York there is an abundance of fresh water, which rises and falls to a greater or less extent with the pressure of tide water upon it. The supply from springs must be very great, because there are a vast number of driven wells throughout the region, supplying water for steam and other purposes. The water level at any one point may be materially lowered at any time by pumping from a driven well in the vicinity or from the constant drainage of some leaking basement or other excavation. Thus it would seem that the permanence of any given water level in the city can rarely be relied upon.

In considering the provision of an adequate foundation for a building, the weight of which seems to require something better than the old style of simple foundations, the first natural thought is to increase in some way the spread of the foundation so that the unit weight on the material may be kept within allowable limits. In Chicago, where

the material on which the city is built consists generally of a thickness of 40 to 100 ft. of blue clay of more or less yielding character which can scarcely be loaded safely 2 tons per square foot, the general practice has come to be that of proportioning the area of the column foundations in a uniform ratio to the column loads upon them, in order to get equal settlement under all the columns. This has resulted in the so-called raft system of foundations, in which, by alternate layers of steel beams of lessening length, it is practicable to spread the bearing to a sufficient extent. This method requires the minimum depth of excavation and gives the minimum amount of cellar space. Short of some method of carrying the bearings to a lower and more substantial stratum, as by piles or by excavation to bed-rock, this would seem to be the best possible treatment of the problem under such conditions as exist at Chicago. That this method should be at all satisfactory requires a comparatively uniform condition of material under the entire building, uniform in character, in compressibility, in softness and in depth.

This raft method has been employed in some cases in New York. For buildings in the lower part of the city it is open to the objection that the conditions under the entire building are not apt to be uniform, and there is often the very serious objection that at some part, if not under the entire building, the material at some depth may be sandy, or of the nature of quicksand, and may yield or be relieved at some future time. Of course, there is always the question as to the durability of steel beams, even though encased in concrete, where they are below the water level.

In considering the loads to be provided for by the foundations for the Manhattan Life Building it was found that if piles were driven over the entire space to be covered by the building, enough piles could not be driven on the lot to sustain the building and load the piles within the limit permitted by the building law. Thus, unless the simple method of spreading the foundations was considered satisfactory, which in this case was out of the question, there was no way but to carry the column loads from the bed-rock, which was here 54 ft. below the Broadway street level. The borings taken on the lot indicated the existence of more or less quicksand, and it was feared that any effort to excavate for foundations in the open to the bed-rock would result in an inflow of material that would endanger the adjoining build-

ings. For this and other reasons, the architects, Messrs. Kimball and Thompson, decided upon the use of steel caissons sunk by the pneumatic process, and thus came about the first use of this method.

Its advantages are: First, the excavation can be carried on under pressure, which holds back from inflow any outside material; second, any obstructions met with in the sinking can be removed without serious difficulty or delay; third, the bottom can be examined, and, where necessary, leveled or stepped; fourth, the work can be executed with speed, because the caissons can be brought on the lot finished or ready to put together, and while these are being sunk the brickwork to form the permanent pier can be built upon them, so that when the excavation for each foundation is completed, the pier itself is finished. This method has been employed in obtaining foundations for the American Surety Building and for the new Standard Oil Building, and its use is contemplated in other buildings soon to be erected. It would not seem worth while to deal in detail with the construction and sinking of the caissons, because the general method is familiar through its application to bridge foundations. Some points, however, should be mentioned. In these comparatively small caissons with vertical sides, sunk the entire distance through earth, the friction is in much greater proportion than occurs in bridgework. As soon as the brickwork, built as high as is practicable, proves of insufficient weight to make the caisson follow the excavation against the upward pressure of the air and the friction, it becomes necessary to put on a temporary load. This is done by piling on pigs of iron. Another respect in which this work differs from bridgework is the fact that excavations in the material should preferably be carried on by means of a Moran or other lock without permitting reduction of the air pressure.

Of course when the depth to hard bottom is not great and there is no running material over it, the excavation may be made in the open air under the protection of sheet piling, and this is in such cases the simplest and cheapest means of doing the work.

The foundations for the new Johnston building have been successfully put in by means of open wrought-iron cylinders sunk by the water-jet process. This method, while somewhat cheaper than the pneumatic, would seem to be open to the objection that it might not permit in some cases a proper examination and preparation of the bottom and of building the foundation in the dry, and, too, should flowing material under

heavy pressure from beneath an adjacent building be encountered, a possibly dangerous vent might be afforded. As yet the pneumatic process is the one safe and sure method for deep excavations, by which all dangers of quicksand or other difficulties can, with certainty, be quickly overcome and a perfect foundation constructed, and this, too, at a cost, where the conditions are determined, which can generally be estimated with comparative certainty.

While the necessity of better practice than has heretofore obtained in building foundations for heavy buildings is happily becoming somewhat more apparent, it is hard to understand the general lack of solicitude on this point on the part of those constructing new buildings. They are generally content to let the judgment of their architect govern, who probably has never before had such a problem to solve. Having engaged him they fail to see the necessity of employing in addition an engineer expert in foundations, and yet the writer ventures to say that in most cases, aside from the other advantages, his fee would be more than saved by the economy resulting from his guidance. It is probable that a sum not exceeding 3 or 4% of the cost of the entire building, added to what the cheapest possible shallow foundation would cost for one of the very high buildings, would cover the extra cost of carrying its foundations to the solid rock, when this is within 70 or 80 ft. of the surface. In many cases this extra cost would be more than offset by the value of the additional story or stories that could be provided beneath the surface.

When an owner has once realized the importance of getting his foundations on the rock he generally approaches the business treatment of the matter in a very narrow-minded and stupid manner. With a distrust born of the imposition of conscienceless contractors, he wants to be certain that his foundations will be completed for a given fixed sum, and this in advance of the determination of the various depths, corrected weights, dimensions and so forth, which is obviously impossible without injustice either to him or his contractor. The proper way for this work to be done is for the owner to engage a party whose integrity and capacity he can trust to do the work for him, for its net actual cost plus a fee as compensation or profit. Such an arrangement brings all interests into unison, permits much preparatory work to be done before the work could be let by contract, and gives the work finally to the owner at what, and only what, should be paid for

it. It results in the best work carried out in the quickest time, with freedom to improve plans as the work progresses, and with the pleasantest relations of owner, architect and foundation builder.

The author wishes to apologize for the absence of detail in this paper; it was intended to present only general considerations. The time would seem ripe for discussion of the matter of foundations of buildings. It should be brought to the minds of architects and property-owners that soils, like metals and woods, have limits of strength and cannot be overloaded without inviting trouble. If there is an absence of that principle to do things well, which should be an instinct with all who build, they must be shown the false economy of their inadequate methods.

This general consideration of the subject treated has been offered in the hope that it will call forth valuable discussion, and that its author is not mistaken in assuming the Society to be the center from which such good influences as are needed must emanate.

DISCUSSION.

Mr. Hains.

PETER C. HAINS, M. Am. Soc. C. E.—The paper refers to foundations of heavy buildings in New York City, but as pile foundations are not uncommon there, and as it is generally believed that piles cut off below the level of standing water are safe from decay, it may not be irrelevant to call the attention of the Society to one case where this rule has not apparently held true.

Between the years 1833 and 1849 several thousand piles were driven for the foundation of old Fort Delaware on Peapatch Island in Delaware River. These piles were cut off about 20 ins. below the level of mean low tide, the mean rise and fall of which is about 5 ft. It is proper to remark, however, that the level of standing water within the enclosure of the fort, while it varies somewhat with the tides, does not vary to the same extent, so that the lowest level of standing water where the piles were driven is above that of mean low tide outside. In other words, the piles were cut off considerably more than 20 ins. below the level of standing water, and there is not the slightest doubt but that the level of standing water has never yet been below the tops of the piles, yet some of these piles decayed to such an extent near their tops that a stick could be pushed through the decayed part several inches.

Several were broken off by the comparatively slight strain brought on them by attaching a guy reaching to the top of a pile-driver frame

Mr. Hains. to steady the latter. The decay was apparently confined to the upper 4 or 5 ft. of the pile, at least that was found to be the case with one that was pulled out, and was apparently the case with all that were examined. The decay was mostly confined to the sappy part of the stick; the heart was scarcely affected, and grillage timbers that rested on the heads of the piles were found to be sound. The decayed part was greatest at the top and gradually decreased downward until at 4 or 5 ft. below the top the decay disappeared.

The piles were of rather poor quality Chesapeake Bay pine from 12 to 14 ins. in diameter, and many of them had apparently been injured by hard driving. The rings of annular growth in some cases showed no cohesion whatever. One pile was pulled out and found to be 27 ft. long, well preserved from the point to about 5 ft. below the top. Another stood a pull of 60 000 lbs., and two broke off, with a less pull, where the chain was attached.

The soil into which they were driven consists of a soft, bluish black mud to a depth of about 42 to 45 ft. From that depth to 85 ft. various strata of sand are found, some fine, some coarse. The sand strata contain plenty of water, the level of the latter rising in the well above the level of mean low tide. Below 85 ft. there is said to be compact clay with boulders. The upper strata of mud has a disagreeable odor with indications of marsh gas. The piles had to carry a safe load of about 30 000 lbs. per pile, if the load was evenly distributed. Between the years 1859, when the fort was finished, and 1866, the fort had settled as much as 4 ins.

This experience indicates that it is not always safe to assume that pile foundations for buildings will remain free from decay even if the timber is below the level of standing water.

Mr. Berg. W. G. Berg, M. Am. Soc. C. E.—Many of the principles and methods of construction described by the author with reference to foundations of buildings can be employed very advantageously for general engineering work. Attention should be directed to the desirability of the more general use of iron and concrete grillage foundations for bridge piers and trestle bents located between the running tracks of railways or adjoining some other structure where a deep and large foundation pit would be objectionable. In making comparative designs between the iron grillage foundation and the general method of spreading foundations by means of masonry and concrete offsets, it is surprising to note in many cases, especially where heavy concentrated loads have to be provided for, the decrease in the size of the bed, the depth and the total weight of the foundation. The probable reason that the iron grillage system has not been used more extensively outside of foundations of buildings can be traced to the uncertainty existing in the minds of a great many engineers as to the durability of iron imbedded in concrete. The author admits that there is a question in this respect when the

iron is placed below the water level. In order to make the system more popular and to allow the use of iron in foundation work to be classed as good practice, it will be necessary to prove conclusively to the satisfaction of the most critical members of the profession that iron or steel will not deteriorate in contact with concrete. Mr. Berg.

Charles Macdonald, M. Am. Soc. C. E.—The speaker was of the opinion that the high-water mark was the lowest level that any property owner might be expected to go with his foundations, and that from 3 to 4 tons per square foot, distributed properly over the New York sand at that level, is certain to give good results. There might be a settlement of a small fraction of an inch, such as has occurred in some of the largest buildings in the lower part of the city, which have been founded upon a concrete grillage or on piles; but why it should be considered necessary to change the practice of years to enable any individual to go down to rock for his foundation is a matter that would seem to be beyond the professional point of view altogether; it is a question of economics. If the owner of property believes he can utilize the space from the rock to the sky, at a price which will remunerate him for the expenditure, that is a question of business policy and not of engineering. It is admitted that the author of the paper is well equipped to put in foundations to the rock by the pneumatic process; but the expediency of resorting to the increased expenditure involved in so doing should not be decided upon engineering grounds alone. Mr. Macdonald

George B. Post, Esq.—For many years there was no decision of the courts on the embarrassing connection between 10 ft. depth of foundations and the liability of owners of property. It was not decided legally until recently whether there was a liability on the part of the owner using due diligence and ordinary care in erecting his building for the settlement of an adjoining building, which was due either to the compression of the soil adjacent to his foundation or to the friction of the walls of the two structures. A few years ago, a decision of the Court of Appeals was made to the effect that every lot has an inherent right of support in the adjoining soil, and if that support is impaired, damages must be paid for doing so. Under this law, in one case where the greatest care had been taken in a building to prevent settlement, this nevertheless occurred, and after a long litigation the party who was carrying on the work paid the cost of repairing the adjoining building. Mr. Post.

In a large district near Broad Street there is a true quicksand which moves with great velocity. The speaker once had a large sheet-piled test pit 8 ft. square dug 30 ft. south of the building of Drexel, Morgan & Company, and 80 ft. east of Broad Street. When the pit had been sunk through the overlying material, the man digging it nearly went out of sight in the quicksand, which squeezed out of the hole with

Mr. Post. great rapidity. Fortunately there was a large amount of concrete being made in the neighborhood, and by ramming this into the hole for twelve hours, the movement of the quicksand was stopped. Meanwhile lintels in buildings 200 or 300 ft. distance were reported to have cracked. This is not the character of the soil over the greater part of the city.

Near the corner of Broadway and Ann Street, where the speaker was erecting a high building, there are many lofty structures. The New York Post Office, a heavy granite building, is directly opposite, the Mail and Express and the Vanderbilt Buildings, Temple Court, the Potter, Times, World and Tribune Buildings all stand on sand, to which they transfer a pressure of over 4 tons to the square foot without causing any serious settlement. In the case of the World Building the blank wall on the east side imposes a pressure of 4.7 tons on the soil, and the foundation is loaded eccentrically.

While the speaker was not a great admirer of steel in concrete, the evidence of the last few years seemed to him to prove forcibly that where iron or steel is in direct contact with masonry it is thoroughly protected from corrosion. In pulling down the first Herald Building, which was over 30 years old, he directed the inspector to bring him the worst corroded pieces of iron he could find in the building. There were no bad examples found, and, where the mortar had been in absolute contact with the paint, the paint itself was preserved. All the reliable evidence obtainable goes to prove that cement mortar has the faculty of preserving iron and steel to a great extent from corrosion, and certainly of preserving from injury the paint which it covers. Another point to be considered in determining the relative advantages of the caisson and grillage foundations is that where caissons are used it is generally necessary to carry the weight of the building to them by means of cantilevers or grillages, and it is hard to see how the beams embedded in the concrete of a regular grillage foundation are in any more danger than such cantilevers and small grillages superimposed on the caissons at about the same level under ordinary circumstances.

It must be remembered that good architecture is always good engineering. The reason a well proportioned stone column eight diameters high is graceful is largely because this is about the limit such a column can be proportioned to bear its load properly. Good architecture and construction go hand in hand; where the engineering is bad, the architecture is generally bad, and where the architecture is bad, the engineering is usually worse.

Mr. Cooper. THEODORE COOPER, M. Am. Soc. C. E.—This paper, from one so capable of presenting facts and principles from his experience which would be of value and interest to every member of the engineering profession, is an unsatisfactory one. It speaks vaguely of "overloading

soils," "great pressures" and "danger of lateral flow," but gives no facts or opinions as to the conditions of loading which would cause lateral flow or produce undue loading. It leaves the reader to infer that the only safe or satisfactory foundations for buildings in New York are those on rock. While there are places where important buildings should be carried to the rock, the speaker did not believe it necessary or possible to go to rock in the great majority of cases in that portion of the city lying below Twenty-third Street. It would be a useless expenditure of money. If a material can be found with sufficient bearing capacity to carry safely the weight of the structure, it would be useless to go deeper than is necessary. Mr. Cooper.

In a general way there are four classes of materials within the business district of the city, upon which buildings must be founded. Along the North River front the material is largely silt, which has been sounded in places to a depth of 200 ft. without reaching rock. The dock walls are floated upon this material in a satisfactory manner.

For two and more blocks back from the river there is made ground partially floating upon the silt. Along the old water-courses and over the former ponds and marshes, where the underlying material is silt or soft mud, there is also made ground. The most difficult foundations in New York City are on this made ground. The maximum bearing capacity of such material depends almost entirely upon the floating capacity of the soft silt or mud immediately below the filled material, for in many cases piles cannot be driven through the filled material far enough into the lower and stiffer silt to increase its bearing power. Moreover, the conditions may be changed by the loadings on the neighboring ground. One of the examples of defective foundations named by the author was on ground of this kind, and the failure was due to the filling in of adjacent ground.

Rock can be obtained for foundations over a small area in the neighborhood of Bowling Green, at depths not exceeding 35 ft. Two miles farther up Broadway, about Fourth Street, rock can be reached at about 40 ft. below the street grade; north of this point the rock generally approaches the surface and occasionally crops out above the surface; there are, however, at places deep depressions in the irregularly underlying rock which are filled with mud, sand, clay or artificial filling.

Between Fourth Street and Broadway and Bowling Green, with the exception of the Canal Street district, the backbone of the island for a distance of about 2 miles is sand, varying from coarse building sand to sand of the finest kind. This sand is from 40 to 160 ft. in depth. It is over this district that the principal high buildings have been built and where the important question as to the necessity of going to rock for secure foundations must be considered.

Mr. Cooper. The illustrations of faulty foundations given by the author would have been of general interest if the full facts had been given, but they have no bearing upon the main question, viz.: Can high and heavy buildings be founded securely upon the sand which underlies the principal business district of New York, or should all buildings of this kind go to the rock below, regardless of cost and other conditions?

The answer to this question must largely hinge upon the bearing capacity of this sand and its liability to be effected by future conditions. The author acknowledges that "until recently" buildings in New York seldom put upon the earth supporting them greater loads per square foot than 3 to 4 tons, and the earth held such buildings reasonably well. Plenty of buildings 10 to 12 stories in height have been well supported with loads of 4 tons, and in some cases as high as 5 tons; if these buildings were well supported under these pressures, when using a small portion of their ground area, why cannot higher buildings be equally well sustained by using the same units over a greater area of their base? Buildings 25 stories in height can be built without exceeding 4 tons upon their full base. There are a number of high buildings which rest upon this sand and have for several years without any signs of difficulty. Tests have been made which show that this sand, even of the finest kind, can sustain at the level of high water loads of 6½ tons per square foot under vibratory action without showing the least evidence of movement after the first small initial settlement, and it is possible that even higher loads could be put upon it under such conditions without movement. The building law allows 4 tons; there has been no evidence produced to show that such loading is not safe and reliable. Proof to the contrary must be advanced before question can properly be made in regard to the reliability of such loadings.

That sand is not a reliable material for foundations, when it is possible to have it removed by the action of running water, from the flow of streams, springs or surface drainage, or when it is so quick under saturation as to be forced out by the pressure, is not disputed. Failures of foundations have occurred from this action, but generally the sand in the district under consideration does not flow readily even under pressures as high as 6½ tons. The mean high-water level, which may be considered as the permanent water line, owing to the sluggishness of flow through this fine sand, is about 20 to 40 ft. below the street grade. High buildings need all of this space for cellars and basements for machinery and storage purposes. It can be assumed then that the foundations of high buildings will go as low as this mean high-water line. At this depth there is no flowing water from surface drainage, springs or tidal flow. The only other natural cause affecting the stability of the sand is the flow due to the pressure of the load; therefore, in all general cases, foundations at this depth can be con-

sidered secure under pressures not exceeding the usual practice of 4 tons, unless they are injured by being undermined by the excavations of some more ambitious neighbor. Mr. Cooper.

This leads to the second branch of the paper, viz., that duty to neighbors requires high buildings to be carried to rock, regardless of expense. The existing law and practice for many years in this city has been to require in new excavations that all previous structures founded 10 ft. or more below the street grade should be protected and maintained by the parties making the excavations. For the earlier buildings of few stories 10 ft. was a sufficient depth, and as buildings increased in height the depths have been increased, but no change has been made in the law. It is questionable if a greater legal depth would be in the line of public policy, for it would tend to put persons of moderate means at the mercy of their wealthier and more grasping neighbors.

If the author intends to suggest the change only for high buildings, it is unnecessary, for self interest will compel the owners to go as deep as the high-water line. If any owner desire to go lower, he should do so at his own expense and risk. No new law can effect existing rights under the present law. If any law is to be enacted in reference to high building, such law, in the speaker's opinion, should restrict the height of any building by an equitable consideration of other people's rights, or far below the heights already used.

The author mentioned a building where the pile foundations have been surrounded by a dam and a pump used to maintain the water level about them. As this is a special case, though having no bearing upon the question of suitable foundations, the speaker gave fuller facts in regard to it. This building, supposed to be a first-class structure, was closed out under a mortgage. The new owners found themselves in possession of a pretentious structure requiring very large expenditures to be put in rentable shape. The building was very extensively rebuilt, and desiring to limit the expenditures to a minimum until the rentable value of the building could be determined, the owners accepted, under advice, this method of protecting the piles. In connection with some advisory work on the superstructure, the speaker was incidentally asked in regard to this method of protection and unhesitatingly condemned it; he then learned that it had already been completed.

C. T. Purdy, M. Am. Soc. C. E.—If caissons can be afforded and the money can be spent to support a building on rock by their means, there will probably be no trouble with the settling of the structure afterward, but in Chicago, where the speaker had been engaged chiefly, the cost of caissons prohibited their use absolutely. Chicago foundations settle, some of them an inch or two, and some of them much more. There are a number of old buildings in Chicago which Mr. Purdy.

Mr. Purdy. have been wrecked because of unequal settlement. The old post office is an example, which was built on a bed of concrete spread over the entire area of the lot without any effort to proportion the bearing area of the concrete to the loads. The same unequal settlement compelled the Board of Trade to take the top off the tower of its building, and mar what was the pride and landmark of Chicago citizens. Hence, in constructing new buildings it was recognized that the problem was to make them so they would settle uniformly, and not to prevent any settlement whatever. It has been demonstrated that if loads are calculated and distributed properly, and the bearing areas of the foundations proportioned to correspond with the weights coming on them, so they will balance about their centers of gravity, the buildings will settle uniformly. It is impossible to predict how much the settlement will be, as the data on the subject point to no uniform law or laws in this respect. The settlement in itself is not a cause of injury to the structure, as is shown by a building in Chicago which has sunk 10 ins. without causing a crack in the plaster.

Where grillage foundations are adapted to the conditions of a structure and can be built more cheaply than other types, there is no good reason why they should not be used. Adjoining walls can be cared for, as is done all the time in Chicago. It is an easy and not expensive matter to put such a wall on pins and keep it there until the building has settled, which generally takes but a few months, and the work is not so expensive as going down through 80 ft. of soft clay to solid rock.

William Sooysmith, M. Am. Soc. C. E., has objected to the grillage foundation and uses in place of it a combination of open caissons and piles. It has proved a very difficult matter to put in such foundations without disturbing the surrounding property. The water-jet cannot be used in the soft clay underlying Chicago, and it is a difficult task to push an open caisson through this material, on account of its tenacity and the great friction between it and the skin of the caisson. E. C. Shankland, M. Am. Soc. C. E., has recently tried driving short piles under the grillage and this may prove quite successful.

Steel bedded in concrete has not been used long enough to warrant a judgment as to its durability, but iron has remained in concrete for twelve or fifteen years without injury. It is also to be noticed that after a careful study of American and European construction of this type, it has been decided to build the subways of Boston of steel-concrete construction, the beams to be painted with a mixture of cement and water before being laid in the concrete.

CORRESPONDENCE.

F. Collingwood, M. Am. Soc. C. E.—With the advent of the enormous buildings now being introduced in all large cities, the methods of founding as well as of building must be readjusted to meet the new conditions in both economy and safety of construction. Experience shows that the materials composing the substrata on Manhattan Island are very heterogeneous in character, and while with the moderate loads imposed by the older style of buildings there was but little unequal settlement, under loads of anything like 10 tons per square foot, the settlement will be so great as to become dangerous. In many places the underlying rock is very uneven, and the tendency to lateral movement and splitting of walls is consequently very great. It is manifest that under such conditions the ordinary rule of a definite increase in width of footings for each story of increase in height is not satisfactory or safe; and yet it is still considered all sufficient by most mechanics, because it is so provided in the building laws. Mr. Collingwood.

In sinking the New York caisson of the New York and Brooklyn Bridge, 20 ft. of a very fine running sand was passed through, which did not appear at all beyond a diameter of about 50 ft. In a distance of some 80 ft. the surface of the rock dropped 18 ft. In other parts of the island the declivities in the rock surface are much greater.

The old method of sounding with a crow-bar after excavating to the specified depth must clearly give way to more scientific methods, and no foundation be constructed without an accurate knowledge of the conformation of the underlying rock. From the writer's experience with the water-jet he is inclined to endorse the opinion expressed in the paper, that the pneumatic system is the safer method of founding in all cases where there is fine material liable to run. It has the further advantage of enabling building to be done in the dry in all cases.

George Hill, M. Am. Soc. C. E.—The paper is disappointing, because it treats in a general way of a subject which is a special one to a very unusual degree, and demands for its proper solution numerous specific examples as to cost of execution and time required for it, in order to enable the consulting engineer or architect to arrive at an approximately correct solution. For five years past the writer has been of the opinion that for the general case, some form of spread foundation was all that was required for the support of a properly designed building. When the first experiments were under way with the pneumatic foundations, he expressed the opinion that they were objectionable on the score of excessive consumption of both time and money without real compensating advantages, and judging as an outsider from the reported prices paid for the execution of the work, the months required for its execution, and the damage done to adjoining Mr. Hill.

Mr. Hill buildings, there has been no reason for a change of view. The cost of execution of the work, the causes for what appear to be interminable delays, the period when the walls of adjoining building cracked by settlement, are all factors which might modify the judgment were they correctly stated, and consequently a statement by the author as to what he had done and the cost of doing it, and the various circumstances attendant thereon, would have been of great value.

A building is erected to serve almost any conceivable purpose other than solely to show the engineering skill of its designer. As a consequence it must have among other elements, and as the most important of them: first, a good artistic effect; second, a minimum of cost; third, a maximum of area available for useful purposes; fourth, a minimum of material; fifth, a minimum of time required for its erection, and sixth, maximum desirability.

These limitations demand therefore for every large building, the complete and skilful use of the skeleton method of construction, by which is meant a steel skeleton secured against all vertical and horizontal forces, to which it may be subjected; encased in a fire-proof covering of ornamental character, the deep reveals being secured by turning the walls in at the windows rather than by thickening them, and the absolute elimination of all self-sustaining walls. The requirements of lighting limit the effective width of base, and consequently the height to which it is possible to carry the building, and brace it properly against wind pressures. Unbraced buildings carried excessively high will, within a relatively few years after they are completed, sway so under wind gusts as to alarm their tenants. The cost of putting in the upper stories and serving them passes very quickly beyond the remunerative point. The writer, having studied the problem carefully, is of the opinion that the economical limit is at sixteen stories, and that as soon as any experience has been gained, this will be abundantly demonstrated.

The author limits the area under consideration to that portion of New York wherein the greater portion of the area has a subsoil composed of a fine micaceous sand. On this the writer has repeatedly founded buildings in which the actual weights ranged from $2\frac{3}{4}$ to $3\frac{3}{4}$ tons per square foot, with a settlement of from $\frac{1}{4}$ to $\frac{1}{2}$ in. The area of the lot, if properly utilized, is therefore more than adequate to carry the superimposed load of a 16-story building, which may be assumed as being 75 lbs. per square foot dead load for floors and framing, 25 lbs. per square foot constant live load, or for a 16-story building a little over $\frac{4}{5}$ ton per square foot. If the walls are averaged at 220 lbs. per square foot 200 ft. high, there would be 22 tons per lineal foot of wall in addition, or about 7 ft. of extra width following around the wall lines. With such a construction it is possible to erect a building, and, at an expense of about $2 a day during erection, to

support the walls of adjoining buildings and preserve them absolutely against settlement, by wedging up as the new footings go down; this was actually done in the case of one building erected under the writer's direction. Mr. Hill.

When sinking pneumatic caissons, the writer has observed that the adjoining buildings were very seriously injured. In the case of the Consolidated Exchange, there were new cracks of about $\frac{1}{2}$ in. caused by this alone, in spite of the fact that the wall is said to be supported on piles.

In regard to spreading the load over the soil, the writer has made comparative calculations of the relative costs of using the Chicago crib method and the method advocated by him in an article in the *American Architect*,* and found the latter much more economical of metal and money, beside affording a very desirable safeguard against injurious effects of undermining of adjoining foundations.

FOSTER CROWELL, M. Am. Soc. C. E.—Certain points brought out in the paper deserve to be emphasized as matters calling for the earnest attention of civil engineers, not only in New York City, but in all other places. First among these from the professional point of view comes the suggestion of the economical necessity of employing expert civil engineers in order to obtain the best results; the description of the contrary methods now in vogue is accurate and suggestive; the condition portrayed is lamentable; and, as the author states, it is hard to understand. Secondly comes the assumption that the Society is the proper center for good influences, corrective of the bad condition, to radiate from. That idea should be forcibly promulgated without reserve. Thirdly is the importance of the engineer's studying and understanding the surrounding conditions of a property where foundations are to be placed under his direction, and wherever possible advising the client against the adoption of plans that will be the cause of ultimate injury to neighboring property, constituting a claim in equity, possibly, against him. Mr. Crowell.

The author's grave suggestions of the evils of overloading a stratum that rests on softer material, causing lateral displacement or squeezing out under the pressure, and consequent injury to other buildings even at a considerable distance, are not overdrawn. The writer knows of cases in point, and especially of one in which all the buildings on the opposite side of a street for a considerable distance were set in motion and some of them wrecked by such overloading.

The conditions at present prevailing in New York are peculiar and to some extent limit the choice of methods; at the same time the experience is valuable to all. The instance cited where the adoption of a pile foundation was impracticable because the entire building space was insufficient to provide room for the number of piles required under

* Vol. XLII, p. 6.

Mr. Crowell. the building law is suggestive in connection with the remarks of the author in regard to the mistake of assuming the sustaining power of a pile foundation to be in all cases the sum of the safe loads on the individual piles. In the writer's paper on "Uniform Practice in Pile Driving,"* this question of overloading a stratum by driving piles too thickly is discussed, as are also the importance and the method of adjusting the unit support in a foundation to the requirements of a building so as to obtain uniform loading, and the further consideration of providing for the effects of vibrations, of machinery, wind, etc., to which the author refers.

Every foundation forms a special study and it is not exaggerating to say that in many cases it is the most important feature in the entire structure, calling for the very best of engineering talent for examination and advice. The amount of money that owners could have saved in known instances by proper methods, as against the unconsidered means adopted, has been very great, and the same rash waste goes on in numerous cases still, because the owner is not aware of the pecuniary value of the right kind of engineering advice, in addition to the safety and security afforded.

A great mistake often made is to adopt a foundation, based on experience of others elsewhere, before the site is examined to ascertain its suitability. A thorough exploration and examination by a fully competent consulting engineer should precede the architectural plans just as certainly as the search of title is a precedent of purchase, and with the same end in view, viz., to guard the owner against injury.

It would be a desirable measure for the Building Department to collect systematically and plot data from every building excavation and boring, of the geologic, topographic and hygrometric conditions of the strata encountered, so that in course of time there would be built up a complete underground map of the city, for the use of all. Examinations disclosing suitable founding strata are as important as those developing the contrary.

Mr. Thomson. T. Kennard Thomson, M. Am. Soc. C. E.—As the author points out if the foundations of a building are put more than 10 ft. below the curb, the owner is responsible for the safety of the adjacent walls, even if they are badly constructed or on entirely inadequate bearings. Sooner or later the owner of some of these insecure buildings will find that his confidence has been misplaced, for the construction of the foundations for a new building, two or three lots away, will draw the water and sand from under the over-loaded foundations, when a bad collapse will be certain, and even the present building laws will not help him.

Would not some such clause as the following be just to all parties:

The owner of any building over, say, seven stories high, the foundations of which are not carried to rock by means of piles or caissons,

* See *Transactions*, Vol. XXVII, p. 99.

must take measures to render his own building safe whenever excavations, etc., are carried on in adjacent or near-by lots. Mr. Thomson.

Beside the objections of decay, etc., raised by the author against pile foundations in the lower New York quicksands, there is a great difficulty in making sure that the piles are driven to rock, for even a good engineer will often have his doubts on this matter, to say nothing of a contractor cutting off the piles too soon when not watched.

There is at least one modern high building in lower New York on a concrete foundation which is so heavily over-loaded that no reliable contractor would care to put in the foundations for buildings on the adjacent lots. The author is quite right in saying that it is true economy to go to rock with a pneumatic caisson for high buildings in a case like this, for no reliance can be placed on a rock foundation unless it can be inspected personally.

The writer thinks that the author is wrong in advocating a force account, for all contractors have not his honesty and ability, nor can even he be sure that his men will always be as honest and capable as himself. The following figures give the cost of some work done under a force account contract:

Material in coffer-dams	$22 667
Concrete	9 172
Plant	7 579
Extra work	88 737
Total	$128 155

On the other hand, the writer has known cases where the contractors paid out from 20 to 40% more than they received, which better management might have saved.

The New York building laws allow at present 20 tons on a timber pile not less than 5 ins. thick, resisting power not stated; 15 tons on solid rock, and 10.8 tons on concrete. As the author states, the load on the piles would be all right if the piles were big enough and long enough, but even poor rock or concrete ought to be good for as much.

The writer recently crushed four 6-in. cubes of concrete nine months old. All were made of one part of Portland cement, two parts of clean sand and four parts of trap rock passing a 1½-in. ring. The first was loaded over its entire surface, and when the load reached 39 000 lbs. the sides cracked off, leaving a block of concrete 4 ins. square by 6 ins. high, which finally collapsed at 104 800 lbs. or 2 910 lbs. per square inch. The second, third and fourth had the load concentrated on a 4-in. block or 16 sq. ins. The second broke at 69 640 lbs. or 4 350 lbs. per square inch; the third broke at 84 300 lbs. or 5 270 lbs. per square inch, and the fourth broke at 87 590 lbs. or 5 470 lbs. per square inch. The first two were of a well-known European Portland cement, and the last two of an equally well-known American Portland.

Mr. Thomson. These cubes were made by the writer while employed as first assistant engineer by Messrs. Jacobs and Davies, chief engineers of the New York and Long Island Construction Company and Bridge, and through their courtesy he was allowed to make the test.

Mr. Harris. R. L. Harris, M. Am. Soc. C. E.—When the author's company built the foundation for the Manhattan Building in New York City, the first use of pneumatic caissons for this purpose, the work marked a long step forward and solved many interesting questions to the great advancement of securing foundations for heavy buildings, as also in other directions.

The author states:

"In one case in lower New York where a very massive and well-built structure resting upon gravel and sand cracked after it had been in use a considerable time, the only new condition introduced to which it seemed possible to attribute the settlement manifested was pumping clear water from a nearby driven well. It seemed reasonable to assume that the water flowing through the sand made possible a slight readjustment of the particles which settled into closer union under the vibration of the machinery in the building."

Another reason may be given, which has been little considered. Water occupies space in the ground and is a supporting element practically incompressible; when excavations are made below the ground water level, as for tunnels, deep cuts, sewers, etc., the ground frequently cracks and opens some distance away, in cases over 100 ft., endangering structures. The previous level of the ground-water has been lowered by the draining and pumping, and this evenly distributed supporting medium has been withdrawn and its bulk displaced, except as elastic air may fill the voids. A notable instance of this on a large scale occurred recently at New Orleans. Care, therefore, should be taken in sinking foundations below those of adjoining buildings that much of the water from their supporting ground be not abstracted.

The author also speaks of the dangerous effect of lowering the level of ground-water in the case of pile foundations; this is sometimes dangerous when the adjoining buildings are built directly on sand or gravel.

In regard to the hydraulic method of sinking foundation caissons for the Johnston Building in New York, it should be remembered that this is the first prominent example of such work for heavy building foundations in the vicinity. Caissons sunk by water have been invented within the past two years from which the possible objections stated are eliminated.

Mr. Hunt. Randell Hunt, M. Am. Soc. C. E.—If the foundation of a heavy building is regarded as an engineering problem, there should seldom be any error in its general design, for an independent investigation would be made of each site and the correct application of engineering principles would determine the character of construction necessary.

Hard and fast methods will not apply to foundation practice, whether for a building or a bridge, and the situation must always be studied with regard to the character of the soil and its liability to change in the future. Serious errors in overloading soils have been made in the foundations of many large structures where they have been founded directly upon wet loam and clay. An investigation of a large number of failures of foundations throughout the world has resulted in this conclusion, and has shown that this bad practice is still carried on in certain parts of this country. Such soils have not the supporting strength usually ascribed to them. Mr. Hunt.

From this fact it should not be inferred that a heavy building cannot be placed directly upon such soils, but simply that the pressures must be reduced considerably below the apparent strength of the soil when first tested and investigated. Although a stratum of more or less firm earth often overlies a softer, watery material, yet the pressure transmitted to the latter may be, and often is, beyond the amount which it can carry safely. In the writer's opinion this is without doubt the chief cause of the settlement of many Chicago buildings, viz., the compression of the softer underlying soil, although the load upon the top stratum might be well within its own supporting power if it were of greater depth.

There have been comparatively few failures of foundations placed upon confined sand. If there is every reason to believe that no extraneous conditions are likely to arise, such as deep excavations in the neighborhood or the flooding of the site with water, then it is seldom that a building is constructed, the foundation of which can not be designed within such limits as to give perfectly safe pressures upon the sand. When such modifying conditions are likely to arise, however, and it is the duty of every engineer to investigate these, then recourse must be had to other methods, but whether the end must be achieved by open cylinders, pneumatic caissons, or piles, appears to be a question more or less undeterminate in advance of a thorough investigation of the physical characteristics and permanence of the foundation site.

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No. 781.

THE TRANSVERSE STRENGTH OF BEAMS AS A DIRECT FUNCTION OF THE TENSILE AND CRUSHING STRESSES OF MATERIAL.

By M. LEWINSON, M. Am. Soc. C. E.

PRESENTED APRIL 1ST, 1896.

WITH DISCUSSION.

The theory of the transverse strength of beams of any shape is based on the assumption that the intensity of fiber resistances is directly proportional to the respective distances of the fibers from an axis called neutral, and that the sum of all the fiber stresses above the axis is equal to the sum of those below. Hence the neutral axis must pass through the center of gravity of the beam section, and, as a consequence, the maximum fiber stress in beams of unsymmetrical section is at those extreme fibers which are most distant from the neutral axis. A further consequence of this theory is that no matter what

may be the material and section of the beam, its transverse strength in regard to the neutral axis is the same, whether the extreme fibers parallel to this axis are subjected to the one or to the opposite kind of forces. It is also maintained that beams of a material whose resistances to crushing and rupture are not equal must invariably break first at the weakest fibers.

It is well known how unsatisfactory are the results of the above theory, which are substantiated by experiments in only a few cases. It is known, for instance, that cast-iron beams of angle or T section are stronger when the compression forces are acting in the webs instead of in the flanges, and that in breaking timber beams the fibers in tension rupture before the compression fibers are affected, although the tensile strength of the wood may be four or five times as great as its compressive strength. This last phenomenon, as also some experiments made on beams of different materials, leads the author to the conclusion that the lower and upper fibers of beams under transverse breaking load are strained to their maximum fiber resistances. This is the fundamental principle on which the following theory is based.

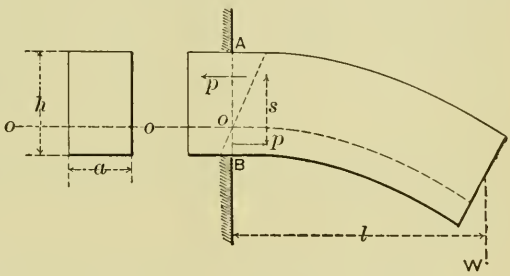


Fig. 1.

Let Fig. 1 represent a rectangular beam fixed at one end and bent, say to the breaking point, by a force W attached at the other end. The weakest place of such a beam is at the plane $A\,B$, which, under the influence of the moment Wl, will rotate around a certain axis O. This rotation is opposed by $p\,s$, consisting of a leverage s and the sum of all the fiber resistances p, above or below the axis O, along the plane $A\,B$. The question arises what is the intensity of p, and what is the relation and position of s to the height of the beam section and its rotation axis respectively? Let x denote the distance of the rotation axis from the lower compressed fibers, the distance of the tension fibers will be $h - x$. The fibers at A are strained to their utmost capacity, say with a tensile stress T pounds per square inch, and the fibers at B are also strained to their utmost capacity with a crushing stress of C

pounds per square inch. These forces, being maximum at the extreme fibers, diminish in intensity toward each other in a direct ratio to the distances of the fibers from the rotation axis, where they meet and are zero. Therefore they can be represented graphically as areas of triangles, the bases of which are $T\,a$ and $C\,a$, and the heights x and $h - x$ respectively. The sum of all the forces represented by each of the triangles being the forces of a couple, they must be equal to each other, i. e.,

$$\tfrac{1}{2}\,C\,a\,x = \tfrac{1}{2}\,T\,a\,(h - x).$$

From this equation it will be found that:

$$x = \frac{T}{C+T} \cdot h \qquad (1)$$

the fundamental expression for the position of neutral axis for rectangular beams. Making $C = T$ in the above formula, as is nearly true in wrought iron, x will equal $\frac{1}{2}\,h$ and coincide with the axis through the center of gravity. For cast iron (tension, 20 000; compression, 100 000), $\frac{C}{T} = 5$; therefore $x = \frac{1}{6}\,h$. For yellow pine, $\frac{C}{T} = \frac{1}{2}$, and $x = \frac{2}{3}\,h$.

The foregoing shows that the position of the neutral axis in a rectangular beam is nearer to those extreme fibers whose resistance is greatest.

It is further evident that the resultant of all the fiber resistances will act at the center of gravity of the triangle representing them, i. e., at one-third of the height of the triangle, or at two-thirds of the distance from the apex O; therefore $s = \frac{2}{3}\,x + \frac{2}{3}\,(h - x) = \frac{2}{3}\,h$, or the leverage of a couple in a rectangular beam is always two-thirds of its height, and its position in the section of the beam is dependent on the position of the neutral axis. This proposition is identical with the general mechanical law that the resultant fiber resistances act at the antipoles of the neutral axis in regard to the ellipse of gyration in the parts of the beam section above and below the axis. As will be seen later this general mechanical law is also true in this new theory for all sections, except that the neutral axis does not always coincide with the axis through the center of gravity.

Having found the magnitude and position of p and s, it is now possible to calculate the transverse strength of a beam, knowing only the crushing and tensile resistances of any material.

Referring again to Fig. 1 :

$$p = \tfrac{1}{2}\, C\, a\, x = \tfrac{1}{2}\, T\, a\, (h - x)$$

and substituting in this equation the value for

$$x = \frac{T}{C + T}\, h,$$

it will be found:

$$p = \tfrac{1}{2}\, \frac{C\, T}{C + T}\, a\, h$$

and the moment will be :

$$M = p\, s = \tfrac{1}{3}\, \frac{T\, C}{C + T}\, a\, h^2 \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

For $C = T$ the moment will be $M = \frac{1}{6}\, C\, a\, h^2 = p\, s$ the familiar formula, in which $\frac{1}{6}\, a\, h^2$ is the moment of resistance of a rectangular beam by the common theory. For glass* it has been found that $C = 30\,150$ lbs., and $T = 2\,560$ lbs., and a 1-in. square bar 12 ins. between supports broke with a center load of 262 lbs. Substituting the values for C and T in formula (2), and noting that $h = a = 1$ in. and $l = 12$ ins., $3\, W l = 786.5$, or the theoretical center breaking weight $W = 262\frac{1}{4}$ lbs., which is identical with the one found by experiment.

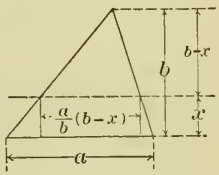


FIG. 2.

For cast iron the author made a series of experiments on beams of various cross-sections, the results of which are appended in Table No. 1. Before explaining these it is desirable to show the derivation from this theory of formulas for other than rectangular shapes.

Let Fig. 2 represent a section of a triangular beam, having a base a, and height b. At the apex the fibers are compressed with a crushing stress C per square inch, and the fibers at the base are stretched with a tensile stress T per square inch. If the neutral axis passes through the triangle parallel to and at distance x from the base, the sides of the triangle will cut from the neutral axis a distance $\frac{a}{b}(b - x)$, and the difference between a and $\frac{a}{b}\,(b - x)$ will be $\frac{a}{b}\, x$. Knowing also that the sum of all the intensities in a triangle in regard to its base is

* See "Strength of Materials," by Thomas Box.

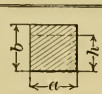
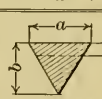
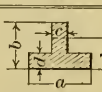
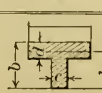
I	II	III				IV	V		VI	VII	VIII		IX
Section of Beam		DIMENSIONS				Clear Span	Ultimate Strength per. sq. inch		Distance of neutral Axis h	Leverage of Couple	Center Breaking Load in Pounds		Remarks
		a	b	c	d		Tensile	Crushing			By Test	By Rule	
	I	1.011"	2.02"			5′0"	19620	81697	1.6288"	1.34667"	1316	1450	Broke in center
	II	0.99	2.00			"	18772	79491	1.6179	1.33333	1165	1323	Broke $2\frac{3}{4}$" from cen.
	III	1.012	2.02			"	21560	83006	1.6035	1.34667	1446	1585	Broke in center
	I	1.836	1.862			4′0"	17676	75954	1.51048	1.24133	2400	2535	Broke $1\frac{1}{4}$" from cen.
	II	1.860	1.865			"	17270	75590	1.51466	1.25333	2321	2519	Broke $\frac{1}{2}$" from cen.
	III	1.865	1.861			"	17538	74586	1.50672	1.24067	2501	2429	Broke $\frac{1}{2}$" from cen.
	I	2.02	2.04			5′0"	21680	81828	0.91612	1.19958	1346	1364	Broke $1\frac{3}{4}$" from cen.
	II	2.03	2.03			"	22308	77256	0.87962	1.18639	1386	1348	Broke $\frac{1}{4}$" from cen.
	III	2.04	2.09			"	25033	90451	0.92188	1.22513	1725	1640	Broke in 2 places $4\frac{3}{4}$" from center
	I	2.00	2.05			"	18430	75532	0.14717	1.05013	665	760	Broke $1\frac{5}{8}$" from cen.
	II	2.04	2.06			"	22944	89678	0.15421	1.05636	826	969	Broke $2\frac{15}{16}$" from cen.
	III	2.03	2.05			"	23204	88253	0.15719	1.05188	860	962	Broke $\frac{1}{4}$" from cen.
	I	4.06	3.00	0.425	0.350	"	27555	115633	1.413260	2.2399	4770	5822	Broke $\frac{1}{2}$" from cen.
	II	4.115	3.05	0.455	0.370	"	26723	98806	1.312192	2.2422	4520	5839	Broke in center
	III	4.155	3.05	0.430	0.360	"	27041	110475	1.383354	2.2643	4990	5975	Broke in center
	I	4.160	3.10	0.440	0.400	"	25816	115498	3.0217	1.9466	2400	2441	Broke $\frac{5}{8}$" from cen.
	II	4.160	3.05	0.420	0.365	"	23841	109616	2.9789	1.8906	2220	2042	Broke in center
	III	4.150	3.04	0.420	0.370	"	21420	102243	2.97124	1.87753	2080	1826	Broke $\frac{3}{4}$" from cen.

TABLE No. 1.—Results of Experiments on Beams of Various Cross-Sections.

one-third of the area of the triangle multiplied by the unit of resistance, the following equation may be written :

$$\frac{1}{6}\, C\, \frac{a}{b}\, (b - x)^2 = \frac{1}{2}\, T\, a\, x - \frac{1}{6}\, T\, \frac{a}{b} x^2 \ldots\ldots\ldots\ldots\ldots (3)$$

From which $x = \frac{b}{2\,(C + T)} \left[2\, C + 3\, T \pm \sqrt{(8\, C + 9\, T)\, T}\right] \ldots\ldots (4)$

If the bar is reversed so that the crushing stress C acts at the base and the tensile stress T at the apex of the triangle, it is only necessary to substitute in formula (4) T for C, and C for T, to get the value for x, which is :

$$x = \frac{b}{2\,(C + T)} \left[2\, T + 3\, C \pm \sqrt{(8\, T + 9\, C)\, C}\right]$$

If C equals T, x will be 0.219224 b. Having determined the position of the neutral axis and substituting the value for x in formula (3), three distinct quantities are obtained. Since the resultant of the intensities in such a triangle finds its point of application on the line drawn from the apex to the center of the base in a perpendicular distance of one-half the height of the triangle from the base, it is easy to calculate the leverage of the couple. For instance, if $x = s_1$; $b - x = s_2$; $\frac{1}{6}\, C \frac{a}{b}\, (b - x)^2 = p$; $\frac{1}{2}\, T\, a\, x = p_1$; $\frac{1}{6}\, T\, \frac{a}{b}\, x^2 = p_2$; then the moment of resistance will be : $p\, s = \frac{1}{2}\, p\, s_2 + (\frac{2}{3}\, p_1 - \frac{1}{2}\, p_2) s_1$ in which $p_1 - p_2 = p$ and $\frac{\frac{2}{3}\, p_1 - \frac{2}{3}\, p_2}{p_1 - p_2}\, s_1 = s_0$ or $p\, s = p\, (\frac{1}{2}\, s_2 + s_0)$; s_0 and $\frac{1}{2}\, s_2$ are measured from the neutral axis, and they are dependent on the position of this axis or on C and T.

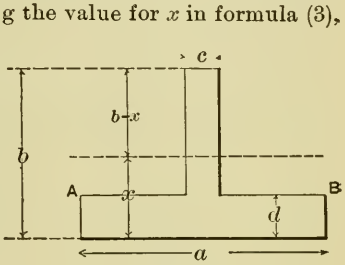


FIG. 3.

The neutral axis in a beam of the section shown in Fig. 3 may pass through the web, or through the flange, or through the edge $A\ B$ of the flanges, i. e., the sum of the intensities in the web may be greater, smaller, or equal to the intensities in the flange. Only the first two cases will be considered, as the third case needs no explanation.

First Case.—Let the neutral axis pass through the web of the beam, and let C be the crushing stress per square inch at the top of web, and T the tensile stress at the bottom edge of the flange; then the unit

of tensile resistance at $A\ B$ will be : $T\frac{x-d}{x}$, and the following equation for x may be written :

$$\tfrac{1}{2}\ Cc\,(b-x) = \tfrac{1}{2}\ Tax - \tfrac{1}{2}\ T\,(a-c)\,\frac{(x-d)^2}{x} \ldots\ldots\ldots \quad (5)$$

$$\text{or } x^2 + \frac{2\ T\,(a-c)\ d - Cbc}{c\,(C+T)}\,x - \frac{T\,(a-c)}{c\,(C+T)}\,d^2 = 0$$

Second Case.—Let Fig. 4 represent the beam section when the neutral axis passes through the flange. If C denotes the crushing stress per square inch at the top edge of the flange, and T the tensile stress at the bottom edge of the web, then

$$\tfrac{1}{2}\ Ca\,(b-x) = \tfrac{1}{2}\ Tcx + \tfrac{1}{2}\ T\,(a-c)\,\frac{[x-(b-d)]^2}{x} \ldots\ldots\ldots \quad (6)$$

$$\text{or } x^2 - \frac{Cab + 2\ T\,(a-c)\,(b-d)}{a\,(C+T)}\,x + \frac{T\,(a-c)\,(b-d)^2}{a\,(C+T)} = 0$$

The determination of the couple leverage s is similar to that in the case of the triangular shape, noting only that for each part of the beam the sum of intensities acts at two-thirds of their heights from the axis. In a similar way, x and s may be found for any shape.

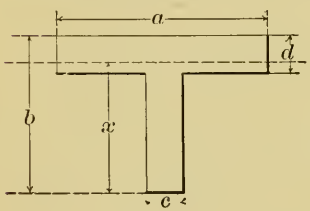


FIG. 4.

Table No. 1 has been prepared according to this theory. Three test pieces for each experiment have been cast from ordinary foundry iron, a mixture of about 50% 2 X Low Moor pig, and about 50% stove and machine scrap. Each specimen with a corresponding mark in column 2 was cast from the same ladle. All the specimens were broken in a 100 000-lb. Riehle testing machine, and tensile and crushing specimens prepared from the broken pieces. For the tensile tests the rectangular and triangular bars were turned down to about $\frac{7}{8}$ in. diameter for a distance of 6 ins., and from the T bars flats were cut about 0.4 x 0.8 ins. for a distance of 6 ins. For the crushing tests, specimens of about $\frac{3}{4}$ x $\frac{3}{4}$-in. base and $1\frac{1}{2}$ ins. high were cut from the rectangular and triangular bars; and pieces of about 0.4 x 0.4 in. base and about 0.8 in. high from the T bars.

While these tests are not all that is to be desired, they nevertheless serve to confirm the theory here presented. It at least clears up to the student of mechanics the difficulties of the moment of resist-

ance and shows that this moment is not always the same in the same beam in regard to the neutral axis. In many text books it is stated that to find the moment of resistance of an unsymmetrical section, divide the moment of inertia for safety by the greater distance of extreme fibers from the neutral axis, implying that the smaller distance would be also correct. This is clearly not true, because from the manner of derivation of the moment of inertia the greater value must be used, and the value so found multiplied by the stress coefficient.

In column 8 of Table No. 1 are given the center breaking loads as found by tests and by calculations, according to the theory here advanced. They correspond fairly well with each other except in the T bar section with the web up, where undoubtedly the wrinkling and buckling stresses added materially to the destruction of the material. The longitudinal and vertical shearing stresses were also not taken into account in these calculations. From equations (1), (3), (5) and (6) it is readily seen that the value of x or the position of the neutral axis does not change if multiples of T and C are taken, showing that these formulas are also correct in case the working stresses are taken in place of C and T. Equation (2) shows also that having two of the three values, C, T, and W, the third can be determined. For instance, a 1-in. square bar of brass, 12 ins. between supports, broke with a center load of 1 150 lbs. The tensile strength of this metal was found to be 17 970 lbs. per square inch. Substituting these values for T and W in equation (2), the value of C is found to be 24 408 lbs. per square inch.

This theory not only gives results more nearly in accord with those found by experiment than the common theory, but gives also a conception of the stresses working in a beam. In designing, it permits the choice of the most appropriate cross-section and material for any given purpose on a rational basis. In closing the author may add that he hopes a better and more extensive series of experiments bearing on this subject will be made on materials more homogeneous than those used for illustration in this paper.

DISCUSSION.

Mr. Marstrand. O. J. MARSTRAND, M. Am. Soc. C. E.—In referring to the common theory of flexure the author states only one of the fundamental laws upon which this theory is formulated, viz., that the intensity of fiber resistance is directly proportional to the distance from the neutral axis, and omits the other, viz., that under small stresses the changes of shape are approximately directly proportional to the forces that produce them; it is these two laws in conjunction that make the stresses above and below the neutral axis directly proportional to their distance. That the sum of the horizontal forces above and below the neutral axis must be zero is a fundamental law of pure mechanics, which is no more applicable to the theory of flexure than to any other. The author omits to state that the common theory of flexure is only supposed to be applicable to stresses within the elastic limit of the material for the very reason that the law referred to, the proportionality of stress to change of shape, or the value of E, is only assumed to be valid for small stresses. The word "small" is of course a relative term, having a different meaning for different materials; for wrought iron it is a much larger quantity than for cast iron and timber. In the latter cases it has been difficult to determine the elastic limit, but for the stresses used in designing in any of these materials, the common theory of flexure is generally accepted as rational. In order to substitute a new theory for the common theory, the author would have to prove, first, that the common theory is not rational within the limit of elasticity, for it is within this limit only that the designer is working, and this the author does not attempt to do at all; secondly, he would have to prove that the theory which he does offer is rational within the elastic limit, and this does not follow absolutely from his assumption and experiments made for rupture only. It is quite true, as he states, that if any multiple of C or T be taken, the value of x in equation (1)* remains the same, but it does not follow that C and T, assumed as maxima for rupture, should decrease in the same ratio, especially when the stresses range below the elastic limit.

Referring to the assumption mentioned, upon which the author bases his theory, the speaker questioned its general applicability, even to cases of rupture, and could not accept, without further argument at least, the hypothesis that the resistances of the lower and upper fibers should reach their maxima simultaneously, no matter what the shape of the section. Neither could he accept the theory that stresses beyond the elastic limit decrease in direct proportion as

* See page 486.

they approach the point of rotation. Referring to the experiments made by the author, it is unfortunate that the discrepancies between the actual and computed stresses should be greatest in the case of the T-shape cast-iron beam, with its flange down, as this is in a modified form, the proper shape to be used for such beams. The point is well taken, that the buckling of the web led to the irregularities, as it shows the importance of having also a small compression flange in such beams. Mr. Marstrand.

H. W. Brinckerhoff, M. Am. Soc. C. E.—According to the assumptions of the author any beam would be as likely to fail by compression as by tension, because the neutral axis so adjusts itself that the extreme fibers on each side are just at the point of rupture at the same time. In fact, all beams should theoretically fail on both sides at once, which does not agree with observed results. It is also difficult to see how, upon his theory, anything is gained by using a T section for cast-iron beams, which is found in practice to be a good form to employ if the rib is in compression. Mr. Brinckerhoff.

W. H. Breithaupt, M. Am. Soc. C. E.—Investigations into the theory of flexure, to be of any value, must be confined to what takes place below the elastic limit. It has been long established that the ratio of stress to strain is constant below this limit, while beyond it this does not hold true. It is therefore futile to attempt by means of observation of what takes place at rupture to establish something governing conditions below the elastic limit. The author's table of results of tests, given in support of his theory, indicates in the large variation in the results of experiments on pieces of similar size the difficulty of securing comparable data, even with the greatest care, with a material so lacking in uniformity as cast iron. Mr. Breithaupt

There is a considerable variation of most of the actual results from those computed on the author's theory. In the group where approximate agreement might be looked for, that of an inverted T section giving compression in the smaller area and tension in the greater, the variations are greatest, being as much at 20.2% in one case. Applying the common formula for rupture,

$$R = \frac{0.75\,(2\,P + W)\,l}{b\,d^2},$$

to the rectangular specimens, the first six of the table, obtaining from the sixth the value of $R = 28\,140$, and using this for the other five, the results obtained for breaking loads are, in the order of the table, 1 272, 1 266, 1 274, 2 470 and 2 501. Although these do not prove much, owing to the unreliability of the material, they are in every case in better accord with the actual results than those obtained by means of the author's theory.

Mr. Breithaupt. The term "moment of resistance" is used in the paper for the moment of inertia divided by the distance to the extreme fiber, $I \div n$. The term has another well-established meaning, $If \div n$. It has been proposed to call $I \div n$ the section modulus and designate it by the symbol S, as is done by Prof. Reuleaux, and this might well be generally adopted.

Mr. Emery. CHARLES E. EMERY, M. Am. Soc. C. E.—The entire discussion has proceeded on the assumption that the longitudinal stresses upon the particles of material at any vertical cross-section of a beam are proportional to the distances of the particles from the neutral axis; whereas it can be proved easily that this assumption is incorrect, as is shown, moreover, by the behavior of beams. In practice a beam will not break until, on this assumption, the theoretical stress upon the outer fibers is double or at least much in excess of that which the material will resist in the testing machine, showing plainly that the portion of the material near the neutral axis carries a larger proportion of the load than its position would indicate according to the common theory. The reasons why this is possible were given by the speaker 17 years ago in a paper,* with illustrations, entitled "Flexure and Transverse Resistance of Beams."

Mr. Waite. GUY B. WAITE, Assoc. M. Am. Soc. C. E.—Mr. Emery's theory shows an accommodation in the distribution of stresses and strains produced during the transverse loading of beams, which is not recognized by the common theory of flexure, while the author's theory shows an accommodation in the location of the neutral axis under such a condition. Both theories give reasonable explanations of the action of materials having unequal tensile and compressive capacities at the point of rupture. It is generally considered that the extreme fiber strains at the point of rupture of rectangular cast-iron beams, as computed according to the usual theory, are about twice the direct tensile strength of the material. Mr. Emery's explanation of the discrepancy is that the distribution of stress through the material is such that the inner fibers receive more than their relative share according to the usual theory, and the author's explanation is that the neutral axis assumes a location giving, in the case of cast-iron sections, a materially increased area to withstand the tensile stresses. Whether or not these theories hold true below the elastic limit, they seem worth consideration when studying the action of materials at the point of rupture in cross-breaking. It is not improbable that, as the adjustment between stresses and strains takes place according to Mr. Emery's theory, the neutral axis is shifted in the manner outlined by the author, so that a complete theory of cross-breaking should take account of both these actions.

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* See *Transactions*, Vol. VIII, p. 149.

CORRESPONDENCE.

S. J. Fortin, Assoc. M. Am. Soc. C. E.—There are many, even to day, who think that the present theory of flexure in beams is incomplete owing to the variations of the coefficient of elasticity E within and beyond the elastic limits. In all American books the writer has examined, E has always been defined as the ratio of unit stress to unit deformation; in algebraical language, denoting by s the unit stress, by λ the unit deformation, this definition would read: $E = S \div \lambda$. If experiments are made on a bar of length l, of section A, and with a load P, the result would be: Mr. Fortin.

$$E = \frac{P}{A} \div \frac{\lambda}{l} = \frac{P\,l}{A\,\lambda} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Another method has been suggested by European writers which is claimed to give closer results, at the same time giving for E a more constant value. It assumes that the increase of deformation $d\,\lambda$ produced by an increment of load $d\,P$ is proportional, not to the original length, l, of the bar, but to the actual length, $l + \lambda$, hence:

$$\frac{d\,P}{A} = E\,\frac{d\,\lambda}{l + \lambda}.$$

By integrating, the following expression is obtained:

$$\frac{P}{A} = E\,[e\,(l + \lambda) + C],$$

in which e is the base of the Naperian logarithmic system:

When $P = O$

$$O = E\,[e\,(l) + C]$$
$$C = -\,E\,e\,(l)$$

therefore

$$\frac{P}{A} = E\,[e\,(l + \lambda) - L\,(l)],$$

and

$$\frac{P}{A} = E\,e\left(\frac{l + \lambda}{l}\right) \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Developing by series:

$$e\left(\frac{l + \lambda}{l}\right) = \frac{\lambda}{l} - \frac{1}{2}\left(\frac{\lambda}{l}\right)^2 + \frac{1}{3}\left(\frac{\lambda}{l}\right)^3 - \frac{1}{4}\left(\frac{\lambda}{l}\right)^4 +, \text{ etc.}$$

Neglecting the factors squared, cubed, etc.,

$$\frac{P}{A} = E\,\frac{\lambda}{l},$$

or

$$E = \frac{P\,l}{A\,\lambda},$$

which is identical with equation (1).

Mr. Fortin. It has always been found that the deformations increase in a quicker ratio than the loads, and that therefore the coefficient E decreases as the unit stress increases. The law regulating the decrease of E is not the same for all bodies. Thus in experimenting on the extensions of bars of different materials tested to rupture, it has been found that for some materials, such as cast iron and cast steel, the decrease of E was slow and regular from the beginning of the experiment to rupture, in such a manner that if the successive unit stresses are plotted as abscissas, and the corresponding values of E as ordinates a curve $a\ b$ (Fig. 6) is obtained which might be called a curve of elasticity, whose minimum ordinate $E\ t$ (value of E' at rupture) is a considerable fraction of $E\ o$ (value of E at the origin of stress).

For other materials, such as wrought iron, soft steel, lead and copper, the decrease of E presents two phases very distinct; the first regular and slow as for cast iron; then a considerable fall is recorded, after which the decrease is again regular and slow until rupture. The curve of elasticity will then appear as $a'\ b'$ (Fig. 7), in which $E\ t$ (value of E at rupture) is only a small fraction of $E\ o$ (value of E at origin of stress).

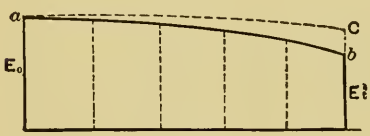

FIG. 6.

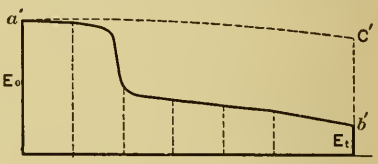

FIG. 7.

In Figs. 6 and 7 the curves $a\ b$ and $a'\ b'$ respectively are the graphical representations of formula (1), $E = P\ l \div A\ \lambda$, in which the deformations are proportional to the original length l of the bar, while in the same figures the dotted curves $a\ c$ and $a'\ c'$, respectively, are the graphical representations of formula (2),

$$\frac{P}{A} = E\left[L\left(\frac{l+\lambda}{l}\right)\right]$$

in which the deformations are proportional to the actual length $(l + \lambda)$ of the bar. The curves $a\ c$ and $a'\ c'$ decrease more slowly than $a\ b$ and $a'\ b'$; moreover $a\ c$ and $a'\ c'$ are similar in shape and equation.

In making experiments, the length of the specimen must be taken into account, as materials do not stretch evenly the total length of the specimen. It is true that the area of the cross-section should not be disregarded, as is the custom at present. It is easy to conceive that after stretching 25% the cross-section of the bar is notably reduced.

Mr. DuBois. A. J. DuBois, M. Am. Soc. C. E.—The author starts with the two assumptions that the lower and upper fibers of beams under a transverse breaking load are strained simultaneously and at the same section to their maximum fiber resistances T and C respectively, and that

the force in any fiber is directly proportional to its distance from the rotative axis. These assumptions seem very questionable. In accordance with them, the author deduces the position of the axis of rotation and the moment of the fiber forces. He thus obtains formulas which give the breaking load in terms of T and C. These formulas he then tests by a series of experiments. He concludes from the results of these experiments that the breaking loads given by his new formulas accord, not only fairly well with the actual breaking loads, but also that his theory gives better results than the common theory. Mr. DuBois.

This is important, if true, and worth examination. Before discussing, therefore, what seems questionable in his assumptions, it would be well to discuss these experiments. A glance at his table of results suggests at once certain points. It will be noticed that although the material was the same in all cases, yet the values of T and C vary considerably in each beam. In applying his formulas the author has the advantage of knowing and using in each case the values of T and C belonging to that case. While this is right in testing his formulas, yet in actual practice he would be obliged to employ a single average value of T and C for all the cases. The table gives no definite information as to how the results thus obtained would compare with experiment, but evidently they would not agree so well. The table thus gives a better showing for the new formulas than would be given by their practical use. The application of this point will be seen later.

The results obtained by the common theory are not given in the table. It is not shown by direct comparison that the results of the new formulas are better than those given by the formula in common use. As this is the entire point at issue, the omission is unfortunate, and the writer computed the strength of some of the beams by the common formula. Time has not allowed him to compute the results for the T bars, but there is no doubt that the same conclusions would hold for them as for the others. The common formula is $\frac{P\,l}{4} = \frac{R\,I}{v}$ where P is the breaking load, l the span, I the moment of inertia of the cross-section with reference to an axis through the center of gravity, v is the extreme fiber distance on the breaking side, and R is the strength of this extreme fiber. From the first experiment R is found to be 28 710 lbs. per square inch, which was used in all the computations. Evidently R might have been found for each case and the average result taken, which would give a better showing for the common method. It has been pointed out previously that the values of T and C in each case have been used in the new formulas, instead of an average value of T and C for all the cases, and this advantage is also on the side of the new formulas. These two points in favor of the new formulas are to be kept in mind while referring to the following table, in which there is given for each experiment the actual strength, the strength

Mr. DuBois. by the new theory and its ratio to the actual, the strength by the common theory and its ratio to the actual, and finally for each case the advantage of each method.

Series.	Experiment.	Strength by experiment.	Strength by new theory.	Ratio to actual strength. Percentage.	Strength by common theory.	Ratio to actual strength. Percentage.	New theory advantage. Percentage.	Common theory advantage. Percentage.
1	I.....	1 316	1 450	110.1	1 316	100.0	.....	10.1
	II.....	1 165	1 323	113.6	1 263	108.4	.....	5.2
	III.....	1 446	1 585	109.6	1 317	91.0	.....	0.6
2	I.....	2 400	2 535	105.6	2 538	105.7	0.1	.....
	II.....	2 321	2 519	108.5	2 579	111.1	2.6	.....
	III.....	2 501	2 429	97.1	2 575	102.9	0.0	0.0
3	I.....	1 346	1 364	101.3	1 342	99.7	.....	1.0
	II.....	1 386	1 348	97.2	1 334	96.2	1.0	.....
	III.....	1 725	1 640	95.0	1 421	82.3	12.7	.....
4	I.....	665	760	114.3	630	94.7	.....	9.0
	II.....	826	969	117.3	710	85.9	.....	3.2
	III.....	860	962	111.8	680	79.0	9.2	.....
						Total.....	25.6	29.1

The tabular comparison shows that the results given by the new theory are not so nearly in accord with fact as the results given by the common theory, even although the value for R is not an average, but is deduced from the first experiment only, and although the values of T and C for each case have been used in the new formulas, instead of an average value of T and C for all the cases. If average values for R, T and C had been taken, the comparison would be still more unfavorable for the new theory.

Case III of the third series, which gives the new theory 12.7% advantage, or about half of the total, is anomalous. The beam broke in two places 4¾ ins. from the center. If this case is neglected, the common theory comes out far ahead, even with the handicaps mentioned. In addition to this, it gives a formula much more simple and readily applied. With this as the outcome of the experimental results it does not seem necessary to discuss the fundamental assumptions upon which this new theory is based.

Mr. Brown. CHARLES O. BROWN, M. Am. Soc. C. E.—The theory of bending cannot be applied in the form the author uses it for stresses beyond the elastic limit, or for materials following no defined laws of elasticity, such as cast iron. Writers on the subject, who credit cast iron with a defined limit of elasticity, place the difference of stress at that limit as two to one, respectively, for compression and tension.

As structures are designed for the purpose of supporting loads without causing permanent change of form, it would be preferable to introduce in the demonstration the difference between compression and tensile resistance at the limit of elasticity. Taking one of the Water-

town Arsenal tests on cast-iron columns, the writer finds a permanent set with a stress of about 3 000 lbs. per square inch, and a modulus of elasticity of 17 600 000 lbs.; in the same test with a stress of 16 000 lbs. per square inch, the modulus of elasticity is computed to be 13 000 000 lbs. This shows what reliance may be placed upon the laws of elasticity when applied to cast iron. Mr. Brown.

When beginning the work on the revision of the building laws for the City of New York, the writer intended to treat cast iron, stone, concrete and other materials not having a defined elastic limit by computing the resisting moments with reference to the extreme compression fiber, and using in this computation a low fiber stress for tension, about 2 000 lbs. for cast iron. This empirical rule would give satisfactory results for members of rectangular section; for designing other shapes, general specifications should guard against mistakes. For an illustration, in a cast-iron beam of the common form of a ⊥ lintel, not more of the width of the bed than would equal one-half the height of the rib should be considered in the computation, and the width of the rib on the top should be made at least equal to one-twentieth of the span.

The writer agrees fully with the author that the neutral axis does not always go through the center of gravity of the section, as insisted upon by most authors, and he has made use of this fact for many years, especially in designing tension and compression members subjected to bending strains in addition to their direct strains. This matter was fully discussed while considering the subject of eccentric loading of columns at the recent meetings for the revision of the building laws.

W. E. Belknap, Assoc. M. Am. Soc. C. E.—Mention was made in the discussion of the paper of the appearance of fractured beams, and conclusions were drawn directly therefrom as to the portions of the section in tension and in compression at the moment of rupture. The writer believes that a fractured specimen has no trustworthy superficial record of the condition of its fibers at that time. It is clear that as soon as the first fibers have given way the original working section has been destroyed, a new and smaller section being the result, and this before actual severance or destruction of the fibers. As fracture continues the working section diminishes, the neutral axis necessarily moving away from the side on which fracture began, and fibers which were in compression at the moment of giving way show at cessation of fracture unmistakable signs of having been finally in tension, or *vice versa*.

It seems natural that experience should show a larger number of fractured specimens to have given way on the tension side than on the compression side. It may be accounted for by the fact that, as the fibers yield on the compression side, fibers back of them are brought into action, while on the tension side this cannot take place, and each

Mr. Belknap. fiber in the line of fracture acts alone. While at the beginning of distortion or deflection the neutral axis may move toward the tension side by reason of the giving way of the exterior fibers in compression, it may reach eventually a point where there is an equilibrium of the forces in compression and tension. Then, when more strain is added, it may move back again as the tension fibers give way, and finally, when fracture has ceased, lie far within the compression side of the original section.

Mr. Prichard. HENRY S. PRICHARD, M. Am. Soc. C. E.—The author calls attention to the fact, well known among those who have to deal with the strength of materials, that the transverse breaking loads for beams cannot be determined with any certainty by methods based on the ordinary assumptions of flexure. This fact is in strict accordance with theory and not in conflict with it, as some suppose, as it is a necessary consequence that rules founded and dependent on the assumption that the elastic limit has not been exceeded are not applicable, except empirically, to cases in which the elastic limit has been exceeded.

The author, in discussing the subject of breaking loads, states in general that the results of "the theory of the transverse strength of beams" are unsatisfactory. He has not explicitly stated that he means this conclusion to apply to the results given by the theory in dealing with working stresses, but as he has given a theory of his own, in conflict with the commonly accepted one, and as he claims that his theory is correct, both for ultimate and working stresses, it must be inferred that he intends his criticism to apply to cases in which the strains are less than the elastic limit as well as to those in which they are in excess.

He bases his own theory on two assumptions :

First.—That the upper and lower fibers of beams under transverse breaking load are strained to their maximum fiber resistances.

Second.—That the stresses diminish in intensity, from the extreme fibers to the rotation axis, where they become zero, in direct ratio to the distances of the fibers from the axis.

The author has not given any demonstration to prove the theoretical accuracy of these assumptions, but seems to rely on certain observed phenomena and on the comparison of the actual breaking loads in given tests with figured breaking loads for the substantiation of his theory.

The phenomena cited admit of another explanation, and in one instance indicate a conflict with, instead of a confirmation of, his theory. This instance is the fact that "in breaking timber beams, the fibers in tension rupture before the compression fibers are affected, although the tensile strength of the wood may be four or five times as great as its compression strength." This fact certainly indicates that the rules founded on the ordinary assumptions of flexure are not applicable to

the determination of the breaking loads of the timber beams in question, but it also shows that in these beams the upper and lower fibers were not both strained to the maximum fiber resistances as his theory assumes. Mr. Prichard

The comparison of the actual breaking loads with figured ones shows considerable variation between them. To aid in the comparison the writer has calculated the percentage of error in the figured results and gives them in following statement :

Section.	First, Rectangular.			Second, Rectangular.			Third, Triangular.		
	I.	II.	III.	I.	II.	III.	I.	II.	III.
Breaking load, test	1 316	1 165	1 446	2 400	2 321	2 501	1 346	1 386	1 725
Breaking load, rule	1 450	1 323	1 585	2 535	2 519	2 429	1 364	1 348	1 640
Percentage difference	+10.2	+13.6	+9.6	+5.6	+8.5	−2.9	+1.3	−2.7	−4.9

Section.	Fourth, Triangular.			Fifth, Tee.			Sixth, Tee.		
	I.	II.	III.	I.	II.	III.	I.	II.	III.
Breaking load, test	665	826	860	4 770	4 520	4 990	2 400	2 220	2 080
Breaking load, rule	760	969	962	5 822	5 839	5 975	2 441	2 042	1 826
Percentage difference	+14.3	+17.3	+11.9	+22.1	+29.2	+19 7	+1.7	−8.0	−12.2

It will be noticed that the errors range from 12.2% deficiency to 29.2% excess, giving a total range of error of 41.4%, and that the average error is 7.5% excess. The author's explanation, in regard to the cases giving the most unfavorable comparisons, to the effect that "the wrinkling and buckling stresses add materially to the destruction of the material" should be given due weight in considering the comparisons.

In the discussion of the ordinary theory and the development of his own, the author has omitted all reference to the relations which exist between distortions and strains. A reasonable assumption as to these relations should constitute the hypothesis upon which any theory of the strains produced by transverse forces should rest. The relations between the intensities of the strains in the different fibers

Mr. Prichard. at their intersections with any given transverse plane, and the position of the neutral axis, are not subjects for independent assumption, but are necessary consequences of the relations existing between strain and distortion. The assumptions made by the author are not consistent with any conception of these relations which it is reasonable to entertain.

Let $A\ C$ and $B\ D$, Fig. 5, represent two transverse sectional planes, which in the free state of the beam are parallel to each other, the plane $A\ C$ being located at the fixed end, and the plane $B\ D$ so near to it that the bending moment at that plane does not differ sensibly from that at the end. Under the influence of the load W, the material will distort, the beam will deflect and the transverse sectional planes will no longer be parallel, the fibers at $A\ B$ will lengthen, at $C\ D$ shorten, at some longitudinal surface indicated as $E\ F$ will remain unchanged, and the ratio between the longitudinal distortions at different distances from $E\ F$ will be directly proportional to such distances. The dis-

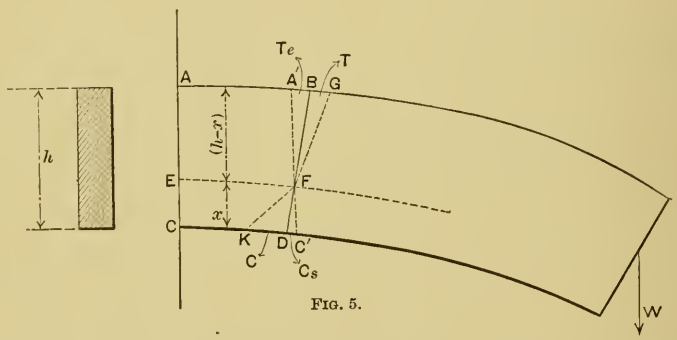

FIG. 5.

tance between the lines $B\ F\ D$ and $A'\ F\ C'$, parallel to $A\ E\ C$, at any given distance from F, is the distortion of the fibers at such a distance. As the fibers at $E\ F$ have neither been shortened or lengthened, the stress in these fibers must be zero and the point F therefore lies in what the author calls the axis of rotation.

From the author's first assumption let the lines $B\ G$ and $D\ K$ represent the maximum fiber resistance in tension and compression respectively.

Let $e =$ the elongation of the fibers $A\ B$, for a tensile stress of 1 lb. per square inch.

Then $Te =$ line $A'\ B$.

Let $s =$ the shortening of the fibers $C\ D$ for a compressive stress of 1 lb. per square inch.

Then $C\ s =$ line $D\ C'$

$$Cs : x :: Te : (h - x) \ldots\ldots\ldots\ldots (a)$$

$$Cs = \frac{x}{h - x}\ Te \ldots\ldots\ldots\ldots (b)$$

Mr. Prichard.

From the author's equation—

$$\tfrac{1}{2}\, C a x = \tfrac{1}{2}\, T a\ (h - x)$$

$$\frac{x}{(h - x)} = \frac{T}{C} \qquad (c)$$

By substituting in equation (b)

$$C s = \frac{T}{C}\, T e \qquad (d)$$

$$s\, C^2 = e\, T^2 \qquad (e)$$

The writer can conceive of no reason why the relation indicated by equation (e) should exist. Substituting in equation (e) the values given by the author for cast iron, on page 486:

$$(100\ 000)^2\, s = (20\ 000)^2\, e \qquad (f)$$

$$e = 25\, s \qquad (g)$$

That is to say, the logical conclusion from the author's assumptions is that in rectangular cast-iron beams the elongation due to tension is 25 times as great per unit of stress as the shortening due to compression, and this conclusion applies, not only to ultimate stresses, but to any stresses, as is apparent from the second assumption, and also from the author's claim that his formulas are correct in the case of working stresses. In the light of present knowledge of cast iron, the assumptions which lead to such a conclusion regarding the relations between distortion and stresses within the elastic limit are not tenable.

From the author's second assumption and the fact that the distortions are directly proportional to their distances from the neutral axis, it follows that the distortion is directly proportional to the intensity of the stress. This conclusion is generally accepted as correct, provided the elastic limit is not exceeded.

At the moment of failure from a transverse load, the fibers at one or both extremes are strained beyond the elastic limit, while those near the neutral axis are not. It is well known that material strained beyond the elastic limit distorts more per unit of stress than material which is strained within such limit, consequently the author's second assumption is not a correct one on which to base a formula for determining breaking loads.

In the foregoing it has been shown that the author's theory is not correct in the case of either ultimate or working stresses. It may be, however, that with some modification to keep their results on the side of safety, his formulas can be used empirically to determine the breaking loads for certain kinds and shapes of beams, as his experiments seem to point that way.

A theoretically correct formula for breaking loads for beams would be an addition to general knowledge and have a scientific interest for engineers, but would not be of much use for determining working loads.

Mr. Prichard. A load which is but slightly in excess of the one which just strains the material to the elastic limit will work permanent injury to the beam, and, if long continued, will probably result in total failure, but it will take a considerably larger load to cause the beam to break immediately after its application. As the consequences of immediate breakage are more to be dreaded than those of permanent injury, the ratio in which the breaking load is required, for safety, to exceed the working load should be considerably greater than the ratio in which the load which will just strain the material to the elastic limit is required, for safety, to exceed the working load. Nevertheless, for beams, the relations between the breaking loads and the loads which will just strain the material to the elastic limit are such that, with the ratios which the judgment of most engineers dictates, it is the load which just strains the material to the elastic limit and not the breaking load, from which the working load is determined. In the discussion of working loads the writer has assumed that the compression flange is stayed against deflection sideways. When beams are not so stayed, buckling, as well as injury and breaking, has to be guarded against, and may become the controlling factor in determining the working load.

Mr. Johnson. J. B. Johnson, M. Am. Soc. C. E.—While the writer has no objection to an effort to show a purely empirical relation between the ultimate strength of materials in tension, compression and cross-bending, he objects strongly to such an effort to place the relationship upon a theoretical basis as has been attempted in this paper. The ordinary theory of the strength of a beam is perfectly correct and entirely applicable to all materials within their elastic limits. It does not hold at rupture for any material which is not perfectly elastic up to rupture, neither does it make any pretense to such an application. Any amount of evidence, therefore, of the failure of the ordinary theory of cross-bending strength when applied at the breaking limit is not to the point. For all purposes for which the engineer has occasion to use these formulas in designing, that is to say, for the greatest working loads which should ever be allowed on any material, the ordinary theory is absolutely correct, and does give the actual stress on the extreme fibers for all such loads. The so-called modulus of rupture in cross-bending, therefore, is well known to be a myth, so far as it is supposed to represent any actual stress on the outer fibers at rupture. It is, however, a perfectly legitimate and convenient modulus to use in designing, since, when the factor of safety is applied, the formula becomes perfectly applicable.

The writer's particular objections to the position taken by the author are:

First.—He assumes without any show of reason whatever that when any beam is broken transversely, the stresses on the extreme fibers on

both the tension and the compression sides of the beam reach their breaking limits simultaneously. That is to say, he assumes that the compressive stress is at its breaking limit on the one side, when the tension stress reaches its limit on the other side. He offers no kind of evidence that this is correct. All persons familiar with the testing of materials in cross-bending are fully aware that this is very far from being correct. In timber, for instance, a beam usually crushes down on the compression side long before it fails on the tension side; whereas, in cast-iron the failure on the tension side occurs long before the material has reached its compressive strength on the opposite side. This assumption most effectually undermines and vitiates the theoretical equations and conclusions which rest upon it. Mr. Johnson.

Second.—In the derivation of formula (1), page 486, the author states that the total tensile stress on one side of the neutral axis must equal the total compressive stress on the other side. In this he is correct, and has not made the usual mistake of persons who object to the ordinary theory of cross-bending strength; but he has made in this connection another mistake which entirely vitiates his equations. He has assumed that the rate of variation of stress on the two sides of the axis is the same, after the neutral axis has shifted from its normal position, which is that of a plane through the center of gravity of the cross-section. That is to say, his a is the same on the two sides of his fundamental equation, which agrees with the stress line shown in his Fig. 1. His figure, however, shows correctly the effect of this assumption, in that it makes the stress area on the compression side of the beam toward which the neutral axis has moved very much less than the corresponding area on the tension side, this being a necessary result of the uniform variation of stress across the section and the shifting of the neutral axis. In other words, these stress areas cannot remain equal under such assumptions (although that they must always be equal is a purely mathematical or geometrical fact), and hence his fundamental equation cannot be true on the assumptions made. The only way in which that equation can be made true is to make the stress vary by a different law on the two sides of the section. As a matter of fact, after the neutral axis has begun to move from its normal position on one or both sides of this plane, the stress no longer varies uniformly, but varies in some one of a number of ways.*

It is well known that the cross-breaking modulus of any material at rupture lies somewhere between the ultimate tensile and compressive strengths, but no theory is competent to determine this relation, since it varies with every different kind of material, and is a function in each case of the particular characteristics of the strain diagrams of

* See "Modern Framed Structures," by Johnson, Bryan and Turneaure, p. 124. This relation is also shown clearly for cast iron in the writer's paper on "Cast-Iron—Strength, Resilience, Tests and Specifications," *Transactions*, Vol. XXII, p. 91.

Mr. Johnson. that material, in both tension and compression, and is mainly dependent on the form of the diagram of the weaker kind of stress. Any such formulas, therefore, as are derived in this paper must be regarded as resting on a purely empirical basis, and without any kind of theoretical foundation.

Mr. Lewinson. M. Lewinson, M. Am. Soc. C. E.—The objections to this paper may be summarized as follows: First, that the formulas are empirical; second, that they do not apply within the limit of elasticity; third, that they give no better results than the common theory.

As to the first objection, if the author's deductions are empirical, why are not the formulas of the common theory also empirical? Is not the common theory based on the elasticity of material presumably derived from experiments? Prof. Johnson calls the modulus of rupture a myth, but nevertheless he calls the common theory perfectly correct. He states further that the author's assumption of the simultaneous maximum fiber stresses is without any show of reason whatever. To this the author would answer that long before he came to the conclusions advanced in the paper, he made many experiments, principally on natural sticks from which the bark had been removed carefully. These were broken transversely around a cushion to eliminate as much as possible any local secondary pressure at the breaking point. In more than 60% of such breakages the tension fibers ruptured before the compression fibers were affected. Of these again the majority split longitudinally in or at the tension fibers. In experimenting with square pieces of timber, similar phenomena were noticed, although in a smaller proportion. In experimenting with beams of brittle materials, he was unable to observe on which side the fibers broke first, as they broke suddenly. Cases are on record, however, where in a cast-iron beam of I section the compressed flange broke first.* On these results the author based his conclusions, and though the experiments were crude, still he thinks he was justified in doing so.

As to the second objection, any mention of elasticity of material was purposely avoided in the whole paper. At first the author showed in Fig. 1 the lines representing the stress area, but he changed it subsequently, as it seemed to him that such a figure would be confusing. He used the rotation plane without any attempt to show how the fibers are stretched or compressed, for the simple reason that he does not presume to know how this is done. He does assert, however, that all the recorded results of experiments on compression under increasing strains are incorrect. They all were made, so to speak, on columns the length of which was too great in comparison with the diameter, and consequently the amount of observed compression was apparent instead of real. For instance a bar 9 ins. long under compressive strain, free to move sidewise 0.01 in. in single flexure, will produce an apparent

* See "Resistance of Material," by DeVolson Wood.

shortening of 0.000003 in. for each inch of column. In double flexure it will produce 0.000013 in. for each inch. In triple flexure it will produce 0.00003 in. for each inch. In following carefully the Watertown experiments on cast iron for the year 1891, it is almost possible to see when a new flexure occurs. For instance up to 4 000 lbs. pressure per square inch no flexure occurred.* The first flexure started at 5 000 lbs., and continued up to 9 000 lbs. Between 9 000 and 10 000 lbs. a new flexure occurred, and at about 30 000 lbs. the triple flexure occurred, which remained until failure. Mr. Lewinson.

The law under which materials compress is not known exactly, and it is just as well to say that the amounts of compression and extension are in inverse proportion to the respective resistances, as to say that they are equal, even within the so-called limit of elasticity.

The author cannot agree with Prof. Johnson as to the shifting of the neutral axis. If it were true that in a cast-iron beam, under a transverse load, beyond the limits of elasticity, the fibers compress more than they extend under the same strains, the axis should shift toward the tensile fibers. Evidently this is not so. On the other hand, if a beam of a material whose resistance to crushing is infinite and whose resistance to tension is finite, is under a transverse load, the neutral axis would unquestionably pass through the outside compressed fibers. If the conditions are reversed, the neutral axis will pass through the outside tension fibers. If one of the two resistances is very great in relation to the other, it is only logical that the neutral axis will pass nearer those extreme fibers having the greater resistance.

As to the third objection, that the formulas give no better results than the common theory, so far as the author's experiments on cast iron show, this is true. Still it is an open question if experiments conducted with a more uniform and homogenous material would not make a better showing. Cast iron was used in the experiments for convenience only, and the author regrets that he was unable to experiment on bars of glass, as at first intended, because of the difficulty of obtaining the desired shapes in this material.

* See "Report of Tests of Metals, etc., at the Watertown Arsenal," 1891, p. 173.

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SCIENCE AND ENGINEERING.

ADDRESS AT THE ANNUAL CONVENTION AT SAN FRANCISCO, CAL., JUNE 30TH, 1896.

By THOMAS CURTIS CLARKE, President Am. Soc. C. E.

The subject of this address is "Science and Engineering," showing the necessary relations between them, which are becoming closer and closer every day, as engineering extends over wider and wider fields of action.

Science is the discovery and classification of the laws of Nature.

Engineering, in its broadest sense, is the practical application of such discovered laws.

In ancient times, knowledge was the treasured secret of a caste. In our day, all can acquire it, and everybody stands on an equal footing.

The education of the engineer is now becoming as broad as that of any of the other professions. Engineering has become a profession, but it is the latest of the professions. A brief description of the evolution of engineering will show why it is so.

The word "evolution" simply means "growth," and there are but few now left who cling to the idea that God created his universe as we see it now. We believe that the world and all that is in it grew from the simplest forms, just as a tree grows from its seed, and the most

highly organized animal from a germ-cell. We see that in all cases this growth is from the simple to the complex, from a state where the parts are few and like each other, to a state where the parts are many and unlike each other, yet all working for a common end.

Not only the universe, this planet, and all its animate and inanimate things, but also all social institutions and professions, and civilization itself, develop under this same law of evolution or growth from the simple to the complex

Herbert Spencer, the great master of this great subject, has shown that our present professions, law, medicine, architecture and engineering, have all developed from the clerical profession. A little reflection will show that this must be so. A profession requires special education and instruction on the part of its professors. In early times there was but one educated class—the clergy—and they were the only ones who knew enough to be lawyers, doctors, architects and engineers.

It is easy to see the connection of law and religion, as law was originally based on equity, and equity and morality form part of religious duty. Disease, in ancient times, was thought to be either an affliction sent by God, or possession by the devil; hence the clergy were the proper medical men.

An art requiring so much technical skill as architecture was originally practiced by the clergy, as temples, churches and monumental structures naturally fell under their supervision. The great cathedrals of the middle ages were built by the higher clergy of those days, among whom William of Wykeham and Abbot Suger are deservedly famous.

A curious survival of this is found among architects even now, who call everybody but themselves "laymen."

The earliest engineers of whom we have any record were the priests of Egypt, whose wonderful works in masonry are surpassed by no masonry works of our days; and the only way in which we can show our superiority is by our greater skill in the use of metals.

Civil engineering did not become separated from the clerical profession until in much later times. We know this from the Latin word "pontifex," which means a bridge builder and also a priest, and from which the words "pontiff" and "pontifical" are derived. The Pope of Rome is still styled Pontifex Maximus, Supreme Pontiff, or chief bridge builder, whichever you please.

The building of a bridge was considered such a remarkable feat, that the name pontifex became the highest clerical title.

Men of inventive talent, not clergymen, displayed much skill in inventing and operating engines of war, hence the name engineer. Civil engineer was a later title, and thus indicates a later development.

Civil engineering is the youngest of the professions, and the reason of its late evolution is this, if engineering be the application of Nature's laws, there could be no engineering profession until these laws were discovered. These discoveries did not come until late in the world's history. It was not until the human mind had become impressed with the constancy of the order of Nature, and that chance had no place in it, that the reign of law began, and men then strove to discover what were those necessary sequences of cause and effect that we call the laws of Nature.

The progress of science was much retarded by the conservative spirit. The works of Copernicus were suppressed. Galileo was imprisoned, and Bruno was burned. Even Newton's splendid discovery of the laws of gravitation was not accepted by those pious persons who feared lest it might detract from the majesty of the Almighty to assume that he worked by law and not by caprice.

Fortunately the world had grown wiser in Franklin's day, and readily believed that the tremendous flash of lightning could be imitated by rubbing a glass bottle with a silk handkerchief, or be drawn from the clouds by a boy's kite.

Men's minds became open to receive knowledge. A common proverb says "One thing leads to another." Hence it is that science in this century freed from the stigma of impiety, and using instruments of the most marvelous precision, has been able to make so many new discoveries.

Some of those which have been made, even in my day, are as follows:

The evolution of life has been shown by Darwin in his doctrines of variation and the survival of the fittest.

The modern geology, which we owe to Lyell, teaches us that this globe came to its present condition by the operation of natural laws still in force, and not by a series of catastrophes. Its vast antiquity, even since the appearance of man, has also been demonstrated.

The researches of Helmholtz, Joule and Faraday have shown the

identity of light, heat, electrical and chemical action as transformations of energy; which has led to what is probably the most comprehensive scientific discovery of this century, the conservation or indestructibility of energy, binding into one law all dynamic action. This shows that power cannot be created, but only transformed, and proves the utter absurdity of Keeley motors and of perpetual motion.

Kirchhoff and Bunsen have given us the spectroscope, which with equal certainty detects the infinitesimal part of a grain of some substance in the flame of a lamp, and also shows the presence of our familiar gases in the light of a star so far away that the ray which now reaches us started on its way before the beginning of Egyptian civilization. The spectrum analysis has created the new astronomy, proving the uniformity of matter, and extending the reign of law all over the visible universe, of which we are but a mere speck.

Such are some of the later discoveries of pure science. Equally wonderful are the discoveries in those branches of science which can be applied to the use of man, and in whose application the engineer takes no mean part.

We have now the new chemistry, which is synthetic as well as analytic. In other words, it not only investigates the composition of bodies, but has learned how to create them by its own processes, such as the invention of aniline dyes, oleomargarine and the economic production of aluminum.

It has revolutionized nearly every art and manufacture. The engineer Bessemer, in the art of metallurgy, has given us the modern steel, which has done as much to lessen the cost of transportation as the invention of the locomotive itself.

Another very important late discovery in medico-biological science is that of the presence of those infinitesimal beings called micobes, which, although so small, exercise such a great influence upon human life and health.

Here the labors of the engineer have come into play and have shown us how to purify water by sand filtration. The experiments of the engineers of the Massachusetts State Board of Health have shown us that sewage, being destitute of oxygen, must be applied to filter beds slowly and intermittently from above, so that air can follow it down and aerate the purifying microbes, and keep them alive to do their work. By this means the worst water can be made chemically pure.

By far the most striking application of science in this century has been in electricity. Morse and Henry have given us the telegraph; Bell, the telephone; Edison, the incandescent lamp; Tesla, the alternating current, and McFarlan Moore hopes soon to give us artificial daylight from glass tubes, in which are no loops nor carbons, but only repeated interruptions of an electric current in a high vacuum. It has also given us the separation of metals from their solutions, and the economic transmission of energy, which has re-created our street railways, machine shops and manufactories, and may soon supersede steam locomotives, where frequent stops are necessary.

Its new discoveries come so fast that one can hardly write them down. Even while preparing this address a great discovery is announced. It has been the dream of investigators to find some means of producing electrical energy from coal without combustion. We now hear of a very simple process by which over 80% of the potential energy of carbon can be converted into electrical energy by causing the oxygen of the air to combine with carbonaceous material, through an intervening electrolyte, preferably melted caustic soda, according to Prof. Cross of the Massachusetts Institute of Technology.*

Assuming that this is true, we may say positively that this discovery will not be of value until the engineer has put it into such practical shape that it will be commercially successful. It may take the place of chemical batteries, but that it can supersede the steam boiler, where great power has to be produced, seems doubtful.

The steam boiler has never had a fair chance until lately. Fuel was crammed under it by ignorant stokers, and the combustion was imperfect for want of oxygen. The scientific engineer has now applied a system of mechanical stoking, which develops much more energy from a pound of coal.

To return from this digression, we may say that electrical discovery has created a new branch of our profession.

The discovery of the X rays is but in its infancy. It can show our bones, but has not been able to penetrate a steel eye bar and show the

* Sodium hydrate is melted in an iron pot and a carbon cylinder is immersed in the molten material. Air in fine jets is blown through the hydrate. The oxygen from the electrolyte combines with the carbon and an electrical current results from the direct oxydation of the carbon, without any material accompanying generation of heat. The supply of air constantly regenerates the hydrate by supplying fresh oxygen to replace that which has entered into combination with the carbon.

bridge engineer any hidden defects. It seems probable that its application to incandescent lighting is near at hand. The engineers are working at this problem.

Little time is left to speak of military engineers, who are well represented in our society. By them the laws of Nature are applied for the defence of mankind, without which the arts of peace would avail but little.

Experience in the last great wars has shown that that nation which has made the greatest advances in engineering is the most formidable in war. It has been well pointed out by the President of the British Institution of Civil Engineers that all improvements in modern instruments of both attack and defence are the work of engineers. High-power guns and armor-plates, repeating rifles and torpedoes were invented by engineers.

Engineers are now developing and improving a very old invention, the submarine boat. Modern discoveries have furnished it with aids which appear to at last make it practicable.

It can now carry its motive power for use in action in storage batteries. Improvements in handling compressed air make breathing possible. Electric wires can control the discharge of high explosives. If its direction can be controlled, it will revolutionize naval warfare.

The battle-ship will have to be designed anew so as to carry several of these submarine boats on her deck. They are torpedoes with men inside of them able to direct them intelligently and evade any nets or other protection that the battle-ship may have.

For harbor defence a larger class can be used.

Before the close of this century we may see another new branch of engineering. Through the labors of engineers man has extended his dominion over both sea and land. Through the labors of engineers he may soon extend his dominion over the air, and aviation engineers may be as numerous as mining engineers now are.

Such are some of the later discoveries of science. Each new discovery leads to still another, and what we know now is as nothing to what we will know in coming years.

All the great discoveries of science have been made by the application of what is called the scientific method of investigation. This is as necessary to the engineer as it is to the investigator of pure science.

Unless he is equipped with it and understands its use he will always occupy a subordinate rank instead of taking his true place as a master of creative science, which I shall show that engineering really is.

Scientific method consists of observation, deduction and experiment.

It has been well said that a healthful skepticism is the parent of observation. Men are naturally lazy, and it is easier to take things for granted than to observe and find out for one's self.

In deduction, we must beware of hasty conclusions. We must be careful not to be deceived into thinking one thing is caused by another because it comes after it. Unscientific persons think that changes of weather are caused by the moon, and bi-metallic philosophers imagine that the kind of coinage with which they buy or sell things affects their value.

On the other hand we must not be deceived into thinking that one thing has no connection with another, because we do not see that connection.

There was a common proverb in England that Tenterden Steeple was the cause of Goodwin Sands, a dangerous quicksand near the mouth of the Thames that has swallowed many vessels. Common people believed this. The wise called it a vulgar superstition, or possibly a coincidence. Lately a scientific observer examined the archives of Tenterden Church, and found there documents showing that a fund, whose revenues had been applied toward maintaining a dyke shutting off a certain arm of the sea, had been diverted by the Abbot of Tenterden to build a steeple for his church. The dyke being neglected for want of money, the sea burst in and caused the Goodwin Sands. So, the vulgar proverb was right after all.

This illustration shows how necessary it is not to draw hasty conclusions, but to sift the facts after collecting them, rejecting those that have no bearing, and taking great care not to overlook or reject those that are of importance.

In forming a theory to account for the phenomenon, one must be very patient and get all the facts first, and not form the theory first and then try and find facts to fit it. This is the pit which scientific men too often dig for themselves and fall into.

The first step, after forming the theory, is to test its truth by renewed observation and by careful experiment, which Huxley calls an

artificial observation of facts. The art of experimenting is not to be learned in a day, but should form the apprenticeship of the engineer.

When a man, having formed a plausible theory, and having found it corroborated by experiment, suddenly runs against some obstinate beast of a fact that upsets it altogether, what shall he do? Shall he abandon the theory, ignore the fact, or go on patiently looking for newer facts? This is the test which shows whether he is imbued with the true scientific method or not.

Cervantes showed great knowledge of human nature when he describes Don Quixote breaking his helmet by a blow of his sword, and then patching it up again. The Don raises his sword to strike it again, pauses, drops his sword, and finally concludes that the helmet is strong enough after all.

True science begins with the use of mathematics and the invention of instruments of precision. Astronomy began to be a science with the invention of the telescope, and engineering with the invention of some kind of leveling instrument.

A science becomes complete when its predictions can be verified.

Electricity became a complete science when Franklin's prediction that his lightning rod would protect from the electric flash was found to be true.

Engineering became a science when the first stone arch carried its load in safety.

How shall our profession be instructed in the scientific method? In past times every one had to find it out for himself as best he could.

Fortunately we have now an abundance of technical schools, in which those intending to be engineers can be trained both mentally and physically in the very things that they ought to know. They can be trained in mathematics and the physical sciences; while in chemical, physical and electrical laboratories they can be taught the essential arts of experimenting. They learn the use of delicate instruments, and the art of drawing.

Already has been felt in our profession the great influx of young men, trained and cultured in all these things. These schools rank as high, and some think even higher, than the older schools of law and medicine.

It must be clear that the value of the training and instruction

which these schools give depends greatly upon the character of their professors. We sometimes hear sneering remarks upon the engineering of professors, when some of them have attempted to design and execute works for which their experience had not qualified them. Clearly, this is the fault of the individual and not that of the class, who are a most able, competent, learned and painstaking set of men. We should look upon them with the highest respect, and aid in every way in helping them to keep in touch with the profession, by sending them copies of our latest plans, specifications and photographs of executed works. We should not be afraid, also, to tell them of our mistakes (for the best of us are but mortals), and aid in filling that vast and useful volume which should contain what we do *not* know about engineering.

Looking at the great numbers of young men who are yearly graduating from the technical schools and entering the ranks of our profession, the fear has sometimes been expressed lest there may not be room for all.

Let us look at the great number of divisions into which engineering, following the general law of evolution, has become, and is still becoming, specialized, and see if there is not yet ample room.

Civil engineering is divided into structural, mechanical, electrical, metallurgical, hydraulic, mining, agricultural, chemical, sanitary, municipal, highway and railway engineering. These classes are again subdivided; as hydraulic engineering into canal, harbor, water supply, power, storage, and irrigation engineering. Railway engineering is divided into bridge, foundation, track, signaling, locomotive and car engineering.

Such specializations come constantly to meet new wants. Thus, we now see developing a special class of engineers created by the great value of land in cities, calling for very high buildings, and requiring experts to design their metallic frames and their foundations, made complex by their limited spaces, and the necessity of avoiding encroachment on others.

The general principles of engineering govern all these special applications, and may best be described in the words of Bacon: "Nature can be controlled only by obeying her laws."

Some persons confuse civil engineering with structural engineering. We read in the prospectus of one of our technical schools that courses

are given in mechanical, mining, electrical and civil engineering. What is meant is structural engineering, or the art of constructing earthworks, piers, walls, dams, bridges, roofs, viaducts, etc.

The term civil engineering has a much broader meaning, and comprehends all engineering except military and naval. It includes all useful works of sufficient magnitude and intricacy to require scientific method, knowledge and skill in their construction. The civil engineer's true position is similar to that of the architect, who commands the services of many different professions and handicrafts. Mechanical and electrical engineers and many others aid him in their several lines, but from him comes the comprehensive design, and he alone is the director-general of the works.

Such is the position claimed for civil engineers in Britain and on the Continent of Europe. The members of our Society should claim it here, and not only claim it, but show that they are capable of filling it. There is but one way in which they can do this, and that is by showing themselves masters of scientific method and knowledge.

In no other way can the vast interests committed to their charge be safely protected. If the engineer has it, and knows how to use it, we shall hear no more of bridges breaking down, dams being washed away, foundations settling, or guessing at the cost of ship canals, those scandals of the profession.

The war between theory and practice has nearly come to an end. We now recognize that either without the other is not of much value. The practical man is one who has had experience in his own special line, and within his narrow limits he is not likely to make mistakes.

The theoretical man without experience is a dangerous guide. Hence we wisely treat the young man, trained only in the schools, as an apprentice, and make him follow instructions until he gets experience. When he has got it, he rises very fast, and soon becomes a master, and in the race of life passes the "rule of thumb" man.

This is the report which we get from the officers of railways and manufacturing corporations, and from chief engineers, all uniting in preferring the trained man.

The progress of civil engineers in the future will be far greater than it has been in the past. Civil engineers will take a much higher rank in public estimation than they have done, and greater emoluments will follow.

Engineering is the great creative science. It has created a new nature governed by Nature's laws, but one which would soon disappear without the support of the engineer. Just as art creates new bodies of men and women, fairer than those of Nature, and literature creates new characters, as alive and real as the people we meet in the street, so does engineering create its new world.

The locomotive is a draft horse, excelling in power, speed and endurance a thousand of Nature's build.

The bicycle is a faster saddle horse, eating nothing but a little oil, and asking only to be kept clean, and that its circular hoof shall be cured of punctures.

The aviation machine, if we get it, will be an improved bird; the ocean steamer is a fish set at work.

The idle river rolls on for thousands of years. The engineer harnesses it with his turbines and makes it work, either on its banks or at a distance through dynamos and wires.

These very electric wires by which we transport sound and energy are nothing but an extended system of nerves.

Great deserts lie uninhabited. The engineer builds his irrigating dams and canals, and the desert blossoms like the rose, with every variety of flower, fruit and grain.

The dreadful pestilences which once devastated the earth have been almost stopped by the sanitary engineer. The engineer has even shrunk up the world itself with his railways and telegraphs. Magellan took over a thousand days to go around the world; when the Siberian railway is completed, it can be traversed in less than fifty.

Shakespeare, with his wonderful prevision, makes Puck say that he will put a girdle round about the earth in forty minutes. This was nearly accomplished at the electrical exhibition in New York on May 16th, when telegrams were sent half around the world and back, a distance of 27 500 miles, in 47½ minutes.

I have thus attempted to trace the evolution of the engineer from the early days when he knew but little (but what he did know he knew well) down to the end of this nineteenth century whose wonderful achievements in all branches of science have made it absolutely necessary that the engineer should be as thoroughly equipped with scientific method and training as are those who have made these discoveries.

Standing, as I do, near the close of my professional career, I address not only to the members of this Society, but to all engineers wherever they may be, this solemn warning—you cannot afford to be ignorant !

When science—the accurate knowledge of what others have done—and experience—the knowledge of what we ourselves have done—are united in the same person, then we may truly say that we have seen the evolution of engineering from a craft to a profession.

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OF THE

American Society of Civil Engineers.

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